ENGINEERING NEWS

A JOURNAL OF CIVIL, MECHANICAL, MINING AND ELECTRICAL ENGINEERING

Vol. LIII. No. 8. New York, February 23, 1905.

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WAVE ACTION IN RELATION TO ENGINEERING STRUCTURES.*

By D. D. Gaillard.†

DEFINITIONS AND THEORY.—*Depth* of water is the distance from the surface to the bottom, when wave action has ceased. *Deep-water* wave is one which is being progagated in water of a depth greater than half of the wave length. *Shallow-water* wave is one in water of a depth less than half of the wave length. It differs from a deep-water wave in that the orbits of its particles are ellipses instead of circles, the eccentricity of the ellipse depending upon the ratio of the wave length to the depth of water. *Wave length* is the distance between the crests of two adjacent waves. *Wave height* is the vertical distance from the crest to the hollow or trough of the wave. *Wave period* is the time between the passage of successive wave crests or hollows. *Wave velocity* is the rate at which a wave advances expressed in feet per second. Theoretically, the wave velocity in deep water is practically the same as that which a heavy body would acquire in falling a distance equal to 8% of the wave length. *Positive wave* is an advancing elevation unaccompanied by a depression.

It is proved mathematically that the elliptical orbit of wave particles is practically a circle when the depth of the water is half the wave length; hence the above definitions of deep-water and shallow-water waves.

HEIGHT AND LENGTH OF WAVES.—The height of a wave depends primarily upon the velocity and direction of the wind and the *fetch*, that is, the maximum distance across open water from the windward shore; but the height is modified by the configuration of the shore line and by the depth of water in which the waves travel. When a wave enters a bay of gradually decreasing depth, its height is sometimes reduced, and the length of crest considerably developed, the wave tending to become parallel to the shore line of the bay. When the wind blows in a direction parallel to the shore, the waves adjacent to the shore swing around and travel shoreward, instead of continuing in the direction in which the wind is blowing. If the shore line then changes so as to lie at right angles to the wind, the waves striking this part of the shore will be much reduced in height, due to dissipation of their energy against the parallel stretch of shore.

Mr. Thomas Stevenson has established, from a large number of observations, the following empirical formula for the maximum height of waves, due to a given fetch:

$$h = c\sqrt{f} \qquad (2)$$

h = height of wave from crest to hollow in feet.
f = fetch in nautical miles.
c = a coefficient varying with the strength of the wind.

In the case of a strong gale, and when the water is of sufficient depth to allow the waves to be fully formed, formula (2) becomes:

$$h = 1.5\sqrt{f} \qquad (3),$$

which is the formula generally used.

Mr. Stevenson considered that for short reaches and violent squalls more exact results are given by the formula:

$$h = 1.5\sqrt{f} + (2.5 - \sqrt[4]{f}) \qquad (4)$$

Formula (3 has its limitations in the case of large areas, for it is not likely that waves much higher than 45 ft. often exist, and these, by formula (3), correspond to a fetch of 900 miles.

On the other hand, it may be doubted whether waves are often subjected to violent winds blowing in a single direction for a distance of 900 miles, ocean gales having generally a rotary motion.

Table II. gives the results of all available observations upon Lake Superior for height of waves.

The fetch in all cases is in nautical miles, and the computed heights in the last column were computed by formula (4) when the fetch was less than 2 miles, and by formula (3) when the fetch was more than 15 miles. It is to be regretted that there were no observations between fetches of 2 and 15.6 miles, as such observations would have aided in determining the limit of fetch to which formula (4) is applicable.

To an engineer a knowledge of the maximum wave height and length is of great importance, for, as shown in equation (1), the total energy of a wave varies nearly as the square of the height, and as the first power of the length. The length and the height of a wave therefore measure its capacity for destruction.

Table III. shows the results of some of the more important observations upon ocean waves in deep water.

In this table for all waves having a height, from hollow to crest, of 40 ft. or more, the sea is described as "heavy." For all the other waves the sea is described as "confused," excepting for the last wave, in which the sea was "regular." [Many other waves of less height are given in the original table from which this table was compiled.—Ed.]

The writer has been unable to find any published data relating to the length or period of waves upon the Great Lakes, and but little relating to their height. Col. T. J. Cram has stated (1868) that the greatest waves at Buffalo, on Lake Erie, were 10 ft. from hollow to crest. Mr. L. Y. Schermerhorn measured (1876) waves 10 to 12 ft. from hollow to crest at Eagle Harbor, Lake Superior. Lieut.-Col. H. M. Robert states that in 1884, at Oswego Harbor, Lake Ontario, during a severe gale from the northwest, the waves attained a height of 14 to 18 ft. above the normal surface of the lake, with a velocity of 44 to 59 ft. per second. As the level of the lake surface during severe storms usually varies considerably from the normal surface, these data can not be used to determine the height from hollow to crest.

In 1901 and 1902 the writer made 650 measurements of waves on 28 different dates, in the Duluth Ship Canal and adjacent waters of Lake Superior. Table IV. [condensed from a larger table.—Ed.] gives the largest waves measured in the given depths of water.

On Sept. 24, 1901, when waves at the outer entrance of the Duluth Canal measured 23 ft. from hollow to crest, there was an opposing current of 2 to 3 ft. per sec., which doubtless increased the height.

As the result of inquiries of vessel captains who have

CONCRETE BREAKWATER AT MARQUETTE, MICH., COVERED WITH ICE.

*Greatly condensed from a paper of the same title, Professional Paper No. 31 of the Corps of Engineers, U. S. A., Washington, D. C.

†Major, Corps of Engineers, U. S. Army.

TABLE II.

Locality.	Fetch.	Height of Wave in Feet. Observed.	Computed.
Duluth Basin	0.375	1.5	2.6
" "	0.428	1.7	2.7
" "	0.641	2.0	2.8
St. Louis Bay	0.916	2.0	2.9
Duluth Basin	0.428	2.3	2.7
Portage Lake	1.086	3.0	3.1
Duluth Basin	0.748	3.8	2.9
St. Louis Bay	1.923	4.5	3.4
San Pedro Bay, Cal.	15.64	6 to 7	5.9
Stannard Rock	41.45	11.0	9.7
Marquette, Mich.	116.63	15.0*	16.2
Portage Breakwater	82.50	16.5	13.6
Duluth Canal	258.62	23.0	24.1
Total		92.8	92.6

*Estimated.

Table III.

Locality.	Year.	Wave. h, ft.	L, ft.	Period secs.	Authority.
So. Pacific...	1885	46.0	765	16.5	Abercromby
Atlantic.....	1848	43.0	559	11.7	Wm. Scoresby
"	1894	40.0	....	....	Officers "Normania"
Wick Bay.....	....	40.0	....	....	Stevenson
No. Atlantic.	1900	40.0	....	....	Vaughan Cornish
So. Atlantic..	1891	39.4	701	11.7	Dr. Schott
Peterhead...	1900	35.0	600	15.0	Wm. Shield
So. Pacific...	....	34.5	....	....	Capt. Chuden
Ind. Ocean..	....	33.6	374	7.5	Lieut. Paris
" ..	1891	32.8	424	9.1	Dr. Schott
So. Pacific...	1839	32.0	380	8.7	Commodore Wilkes
Ind. Ocean..	1892	24.6	1,121	14.5	Dr. Schott

Table IV.

Date.	Height, ft.	Length, ft.	Period, secs.	Mean depth. ft.
May 23, 1901....	17.0	200	8.0	26.7
Aug. 9, 1901.....	14.0	130	6.8	26.7
Sept. 24, 1901...	23.0	250	8.0	26.7
Nov. 22, 1901....	16.0	150	6.4	26.7
April 22, 1902...	13.5	275	9.1	26.7
May 20, 1902....	14.5	200	8.0	26.7
Oct. 23, 1902....	13.0	200	8.0	26.7
Nov. 12, 1902....	16.0	250	9.2	26.7
April 22, 1902...	12.0	195	8.9	18.7
May 20, 1902....	13.0	187	9.0	15.9
April 22, 1902...	10.0	180	8.7	14.0
Oct. 23, 1902....	10.0	164	9.6	14.0
June 5, 1902.....	4.0	73	6.5	7.0
April 26, 1902...	4.5	90	5.6	6.3
June 5, 1902....	3.5	70	5.6	5.7
April 23, 1902...	2.75	70	6.2	4.2
June 3, 1902....	2.5	50	4.8	3.8

made note of the fact that in unusually severe storms the horizon could not be seen from the wheelhouse when the ship was in the trough, it seems probable that deep-water waves upon Lake Superior may be from 20 to 25 ft. high, with a length of 275 to 325 ft. Owing to its greater size and depth, larger waves occur upon Lake Superior than upon any other of the Great Lakes.

REDUCTION IN HEIGHT OF WAVES ON PASSING INTO A CLOSED HARBOR.—Stevenson makes the following statement:

When the piers are high enough to screen the inner area from the wind, where the depth is tolerably uniform, the width of entrance not very great in comparison with the section of the wave, and when the quay walls are vertical, or nearly so, and the distance not less than 50 ft. from the mouth of the harbor to the place of observation, the following formula is applicable:

$$x = h\frac{\sqrt{b}}{\sqrt{B}} - \frac{\left(h + h\sqrt{b}\right)}{\sqrt{B}}\sqrt[4]{D} \quad (7)$$

$$50$$

in which (all dimensions being in feet):

h = height of wave at entrance.
b = breadth of entrance.
B = breadth of harbor at place of observation, or, more accurately, the length of arc having radius D.
D = distance from mouth of harbor to place of observation.
x = height of reduced wave at place of observation.

Stevenson was of the opinion that the preceding formula is of general application in all close harbors where the entrance is of a direct and simple nature, and in which there is no recoil action, produced by walls or obstructions, to the shoreward motion of the waves.

The Duluth, Minn., ship canal and the adjoining harbor basin conform in a general way, though not strictly as regards all details, to the conditions described by Stevenson. In 1902 the writer made observations at various points in the harbor, for the purpose of testing the applicability of formula (7) to the locality in question. The results of these observations were as follows:

TABLE VII.

Station of Observation.	D	B	b	h	Height of Wave. Observed.	Computed.
3	1,200	3,351	300	9.9	1.17	1.45
4	1,300	2,813	300	9.9	2.25	1.66
5	2,000	3,857	300	9.9	2.50	0.95
6	2,000	3,857	300	9.9	1.00	0.95
7	2,930	4,142	300	9.9	2.25	0.81
2	780	1,960	300	11.0	2.26	2.68
4	1,300	2,813	300	11.0	1.74	1.84
6	2,000	3,857	300	11.0	1.65	1.06
8	4,195	6,000	300	11.0	0.45	0.29
1	720	1,809	300	7.0	1.00	1.83
Total					16.27	13.52

The observed wave heights of 2.25, 2.50 and 2.25 were taken at points along the bulkhead wharves which lie so close to the ship canal as to prevent the waves from expanding, hence the great discrepancy between observed and computed wave height. The harbor basin is quite unsymmetrical with respect to the axis of the ship canal. Annoyance from swells in the harbor during storms would have been almost entirely prevented if the city of Duluth had cut the canal farther to the south, thus permitting entering waves to expand freely before reaching the north side of the harbor.

VELOCITY OF WAVES.—The theoretical velocity of propagation of a wave in deep water is:

$$v = \sqrt{5.123\ L} \quad (8)$$

v = velocity in feet per second.
L = length of wave in feet.

The following table [not reproduced.—Ed.] gives the results of observations by 11 different observers on 85 waves on 41 different dates. In 62 cases the velocity computed by equation (8) is greater than the actual velocity, while in the other 23 cases it is less, the average computed velocity being about 9% greater than the average observed velocity.

The theoretical velocity of propagation of a wave in shallow water is:

$$v = \sqrt{\frac{b}{a}\ 5.123\ L} \quad (9)$$

a = semi-major axis of the elliptical orbits of the water particles at the surface.
b = semi-minor axis of the elliptical orbits.

Values of $\frac{b}{a}$ are given in Table I. for corresponding values of $\frac{d_o}{L}$.

The velocity of shallow-water waves is always less than the velocity of deep-water waves. If v = velocity of a deep-water wave of length L, and if d_o = depth of water from the center of the surface orbits to the bottom, then the velocity of v_s of a shallow-water wave (having the same length) may be expressed thus:

$$v_s = k\ v.$$

Values for k are given in Table I.

TABLE I.

$\frac{d_o}{L}$ =	0.05	0.10	0.15	0.20	0.30	0.40	0.50
k =	0.55	0.75	0.86	0.92	0.98	0.998	0.998
$\frac{b}{a}$ =	0.304	0.557	0.737	0.850	0.955	0.987	0.996

The writer made nearly 100 observations upon ocean waves at North Beach, Fla., the observations being taken upon waves just before breaking, the height of the waves varying from 2 ins. to 7 ft. Eighty-four of the observations were made during calm weather, and are given in the following table [not reprinted.—Ed.], from which it will be seen that in nearly every instance the actual velocity exceeded the velocity computed by equation (9), the average excess being 7%.

During the seasons of 1901 and 1902, the writer made 631 observations upon waves near the outer end of the Duluth Ship Canal, and in Lake Superior near the canal. The waves varied in height from 2 to 23 ft., and in length from 45 to 275 ft., and the mean depth varied from 3.3 to 27 ft. The following table [not reprinted.—Ed.] shows that the average observed velocity differed from the average velocity computed from equation (9) by less than 1%, the computed velocities being the greater. This result must be regarded as confirming the reliability of equation (9).

During the progress of wave observations near the Duluth Canal, it was considered desirable to note the effect produced upon wave length and velocity by gradual decrease in depth. The results of these observations are given in Table VIII. The velocities in the tenth column were computed by the following empirical formula deduced by the writer:

$$v_1 = 0.9\ v\sqrt[4]{\frac{d_1}{d}} \quad (10)$$

in which v and v_1 are the velocities of the same wave for the depths d and d_1, the depth d being greater and the water shoaling from d to d_1. It is not probable that formula (10) would apply where d is much greater than $\frac{L}{3}$, nor where the slope of the bottom is much steeper than in the cases embraced in Table VIII. The "mean depth of water," given in columns 4 and 8, refers to the mean depth over the distance in which the observations for wave length and velocity were taken, generally embracing a distance of 200 ft. When a wave reached a depth at any point "b," near the mouth of the canal, equal to the mean depth in the canal, the part of the wave entering the canal traveled on with practically undiminished velocity, while the parts of the same wave in the lake on each side of the canal gradually decreased in length and velocity owing to the effects of diminishing depth. The "Distance traveled by wave," given in column 9, is the distance from the point "b" to the mid-point of observation in shallower water near shore.

PER CENT. OF WAVE ABOVE WATER LEVEL.—For deep-water waves the theoretical relation of the crest of the wave to the undisturbed water level is given by the following equation:

$$a = \frac{h}{2} + 0.7854\frac{h^2}{L} \quad (5)$$

a = height of wave crest above still-water level.

For ordinary waves in shallow water this equation gives values for a that are too small.

The question is of importance to the engineer, for example, in fixing the minimum height for the deck of a wharf or pier supported on open pile work, or in determining the height of a parapet wall intended to prevent waves running parallel with a pier from washing over it. Especially is this the case in harbors like those of Lake Superior, where floating logs and blocks of ice 2 or 3 ft. thick may act as battering rams on a work from the vicinity of the wave hollow to its crest.

It is often erroneously assumed that the true mean level is at the mid height of the wave, an assumption which is far from correct, especially in case of waves in shallow water. Mr. Thomas Stevenson states that at Wick Bay observations upon some of the larger waves during storms showed that about 66% of the height of the wave was above the undisturbed water level. At the mouth of the Columbia River, in water about 26 ft. deep, the crests of waves have been observed 15 ft. above the undisturbed water level. The height of these waves was not noted, but it is fair to assume that they probably did not differ greatly from 22 or 23 ft.

At North Beach, St. Augustine, Fla., the writer made observations upon 45 waves, varying in height from 2½ to 6 ft. The waves were regular swells in calm weather and the bottom was smooth, hard sand, with a uniform slope of 1%. It was found that a varied from a = 0.67 h to a = 0.89 h, the mean value being a = 0.76 h. With a gentler slope or with the wind opposed to the direction of wave travel, a was increased.

During 1901-2, the writer made 616 observations upon waves in the Duluth Ship Canal in water of a mean depth of 26 ft. upon waves which passed without breaking. The following table [Greatly condensed.—Ed.] gives some of the results of these observations:

Table V.

h, ft.	L, ft.	Height of crest above still water. Observed, ft.	Computed, ft.
1.0	115	0.5	0.51
3.0	130	1.7	1.64
5.0	132	2.7	2.88
7.0	140	4.5	4.20
9.0	153	5.9	5.56
12.0	173	8.2	7.86
15.0	205	9.7	9.70
18.0	210	11.2	12.90
20.0	210	13.0	13.81
23.0	210	15.0	16.54

Equation (5), which was deduced for deep-water waves, may be written in the following general form:

$$a = \frac{h}{2} + c\frac{h^2}{L} \quad (6)$$

in which c is believed to be a constant for any particular locality for shallow-water waves not near the point of breaking. The last column in Table V. was computed from equation (6), assuming c = 2, and the sum of all the computed heights differed from the sum of all the 616 observed heights by less than 3%. The mean of all the observations gives a value of a = 0.637 h, showing that nearly two-thirds of the wave height was above the still water level. Beside these 616 observations in deep water there were 173 observations taken in Lake Superior upon waves traveling in shoaling water (bottom slope 1 in 30 to 1 in 90), and breaking after passing from 50 to 100 ft. beyond the point of observation; and most of the observations were taken when strong winds were blowing in the direction of wind travel. The waves ranged in height from 2 to 13 ft., the mean height being 6.7 ft., the mean wave length 132 ft., and the mean value of a = 4.86 ft., so that the mean value of a was a = 0.73 h. This result agrees closely with the observations at North Beach, Fla., where a = 0.76 h.

DEPTH IN WHICH WAVES BREAK.—Due to the action of high winds, currents, and perhaps other causes not so well understood, deep-water waves *may* break, partially at least, in water of ample depth for their free propagation. Therefore, no matter how great the depth of the surrounding water, a barrier opposed to them *may* at times be subjected to the direct action of breaking waves. On the other hand, waves invariably break on reaching water of insufficient depth, and it is therefore of great importance to engineers that they should seek to discover the relation between the dimensions of a wave and the depth in which the wave breaks. For, with this relation established, it will be possible, in many cases, to determine in advance the maximum wave which can assail a contemplated structure. In observations heretofore taken for this purpose, the wave height only has been considered, and this has been compared directly with the depth of the water in which the wave broke. As would be anticipated, the results obtained at different localities do not show a precise ratio between the height of the wave and depth at which it breaks.

On the coasts of the United States the most violent wave action is probably found on the Pacific Ocean, between San Francisco and the Straits of Fuca. The Board of Engineers for the Pacific coast in 1879 collected evidence bearing upon this point, and came to the conclusion that "waves break into combers in depths of 8 and even 10 fathoms of water," although some of the testimony was to the effect that waves had been seen to break in 15 fathoms. Mr. G. B. Hegardt, U. S. Assistant Engineer, states (1902) that in violent storms waves have been observed to break at the whistling buoy over the Colum-

Table VIII.

Duluth Ship Canal. Wave. Length. L.	Wave. Velocity. v.	Mean depth of water. d.	Lake Superior, near canal. Wave. Height. h_1.	Wave. Length. L_1.	Velocity. v_1.	Mean depth of water. d_1.	Distance traveled by wave.	Computed velocity. v_1.
209	28.5	25.8	8.6	174	22.4	18.7	316	23.6
168	27.1	25.9	....	165	21.3	18.4	316	22.4
159	22.9	26.5	7.5	159	19.0	15.7	408	18.1
124	23.4	26.0	6.6	....	18.7	15.3	408	18.5
164	25.7	26.7	7.6	110	20.1	14.0	493	19.9
162	25.0	26.5	6.9	140	18.3	13.7	493	19.1
209	28.5	25.8	7.4	162	21.6	13.5	408	21.8
160	21.1	26.0	6.2	77	....	12.0	800	....
100	21.0	26.6	3.3	69	14.6	6.9	1,200	13.4
70	18.2	25.7	3.0	50	12.5	5.7	1,175	11.3
165	20.0	24.7	3.0	75	12.9	5.6	1,120	12.4
100	21.0	26.6	3.0	50	13.3	5.6	1,220	12.9
100	21.0	26.6	3.0	60	12.8	5.1	1,136	12.5
100	21.0	26.0	2.5	60	12.5	4.2	1,248	11.9
165	20.0	24.7	2.6	65	11.2	4.0	1,208	11.3
130	22.2	25.8	2.5	50	10.5	3.3	1,270	12.0
100	21.0	26.0	2.0	45	9.1	3.3	1,276	11.3
Total					250.8			252.4

bia River bar, in a depth of 15 to 18 fathoms. A wave 50 ft. in height would probably break in a depth greater than 14 fathoms, and *might* break in a depth as great as 22 fathoms. Mr. Thomas Stevenson states that some of the large waves in Wick Bay, during storms, were noticed to break when they came into water of the same depth as their height. He states that waves break in deeper water when the bottom shoals suddenly than when the slope is gradual.

The writer made observations at North Beach, Fla., upon waves varying in height from ½ ft. to 6 ft., before

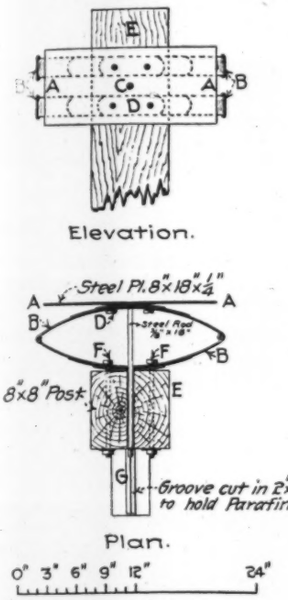

Fig. 1. Spring Dynamometer Used to Determine Wave Pressure at North Beach, Fla.

breaking. The bottom was hard sand, having a uniform slope of 1 in 100, and the breakers were mostly those due to regular swells during perfectly calm weather. It was found that d varied from 0.72 to 2 h.

d = depth of undisturbed water.

h = height of wave from hollow to crest.

For the greater number of observations d was equal to h. With a strong wind blowing in the direction of wave travel, d became equal to 1.25 h; but with an equally strong wind in the contrary direction, d was equal to 0.72 h at the same locality. When there was no wind, the ratio of d to h varied with the slope of the bottom. When the bottom slope was 1 in 100, d became equal to h; but when the slope was 1 in 12, d became greater than 2 h. In all there were 10 observations, in which d was less than h; but in every case there was no wind, or one directly opposed to wave travel.

In 1901-2 the writer made 134 observations upon waves in Lake Superior, 89 of these observations being near Duluth Canal, a few of which are given in Table VI., d being the depth in which waves broke:

Table VI.

h, ft.	L, ft.	Velocity, ft. per sec.	d, ft.	d/h
2.5	....	14.0	3.9	1.56
3.5	75	15.0	7.4	2.11
3.5	63	12.2	4.5	1.29
4.0	82	16.6	7.4	1.85
4.0	70	16.0	5.3	1.32
6.0	164	21.0	15.9	2.65
6.0	94	16.9	9.3	1.55
8.0	180	24.0	20.9	2.61
8.0	163	16.9	13.2	1.65
9.0	180	24.0	20.9	2.32
9.0	114	16.9	12.6	1.40
13.0	....	24.0	17.9	1.38

The mean value of d was 1.84 h. The slope of the bottom was as follows:

d = 2 to 8 ft., slope = 1 in 50.
d = 8 to 12 ft., slope = 1 in 90.
d = 12 to 16 ft., slope = 1 in 40.
d = 16 to 21 ft., slope = 1 in 30.

The average wind velocity was 30.4 miles per hour.

The mean value of the 134 observations was d = 1.67 h. It was found that this ratio decreased with a decrease in wind velocity and in slope of the bottom, and increased with an increase of wave length and of irregularity of the bottom. It appears probable that the undertow also plays an important part in causing waves to break sooner than they otherwise would. When a wave encounters an opposing current its height is increased, its length decreased, and its velocity consequently diminished. The wave front becomes steeper and steeper, and the wave frequently breaks in part or in whole, presenting much the same appearance as if it broke on a bank or shoal.

THEORETICAL WAVE ENERGY.—The energy in a wave consists of the kinetic energy, due the revolution of the particles in their orbits and the potential energy due to the elevation of the center of gravity of the mass of water above its position in the corresponding mass at rest. It is proved mathematically that the total energy of a deep-water wave is one-half kinetic and one-half potential, and that this total energy may be expressed by the formula:

$$E = 8\ L\ h^2\ (1 - 4.935\ \frac{h^2}{L^2}) \qquad (1)$$

the weight of sea water being taken at 64 lbs. per cu. ft.

E = energy in foot-pounds.
h = height of wave in feet.
L = length of wave in feet.

It may be shown mathematically that, during a single complete wave period, an amount of energy equal to half of the total energy of the wave is transmitted forward with the wave form. The energy of the shallow-water wave is slightly less (never more than 11% less, and usually about 2% less) than that of the deep-water wave of equal height and length, the difference in energy depending on the depth of the water. As in the case of the deep-water wave, the energy is half kinetic and half potential, the latter being transmitted onward with the wave form. The kinetic energy undergoes no change, since the particles continue to describe ellipses with unchanged angular velocity.

A *positive wave* consists of an advancing elevation, unaccompanied by a depression. Such a wave leaves still water behind it as it passes, therefore the total energy is transmitted with the wave, and will be delivered at any point, diminished only by the almost insensible effect of fluid friction. When the depth of water decreases gradually and uniformly, the ordinary shallow-water wave tends to assume the characteristics of the positive wave, and may therefore deliver at any point an amount of energy approximating the total energy of the wave.

DYNAMOMETER TESTS OF WAVE FORCE.—The writer has made tests of wave force, using three types of dynamometers, shown in Figs. 1, 2 and 4, which will be described. The dynamometer in Fig. 1 consists of a ¼-in. steel plate, A A, bolted to two carriage springs, B B, placed 6 ins. c. to c. Protruding from the plate A A is a rod, c. The dynamometer is fastened to an upright post, E. The rod, C, passes through a hole in the post, and projects 1 in. beyond, terminating over a groove partly filled with melted paraffin and beeswax, over which a cover was lashed. A small pointer on the end of the rod, C, recorded the maximum wave effect on the surface of the paraffin. Test loads were piled upon the dynamometer to determine its rating. The springs compressed 1 in. for every 120 lbs. of pressure. Two of these dynamometers were used at North Beach, Fla., and the

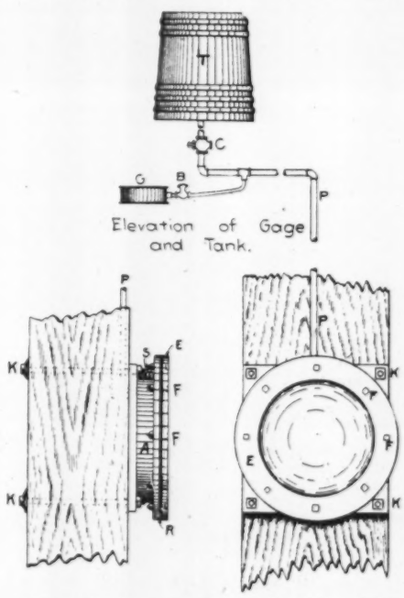

Fig. 4. Diaphragm Dynamometer for Measuring the Impact of Waves.

post was so placed in the line of breakers that they received the full impact of the breakers. Owing to the rise and fall of the tide, all effects of direct wave action occurred between 2 ft. above low water and 11½ ft. above low water. At no time was the depth of water greater than 6½ ft. In all, 197 readings were taken, and Table IX. gives some of the results.

Fig. 2 shows the type of spring dynamometer used on Lake Superior. It consists of a 12 x 12-in. pressure plate, A; a crucible steel spring, B; four recording rods, C, moving in the cavity, G. Rubber buffers, E, take up the shock when the round bolt heads, F, strike after pressure is released. The recording rods were covered with a mixture of melted wax and paraffin, which was scraped off in passing through the cast-iron bed plate, D. A mean of the reading of the four rods gave the maximum wave pressure. Ten of the dynamometers used were of such strength that 1 in. of compression corresponded to 885 lbs. of pressure; in three of the dynamometers, 1 in. corresponded to 1,065 lbs.

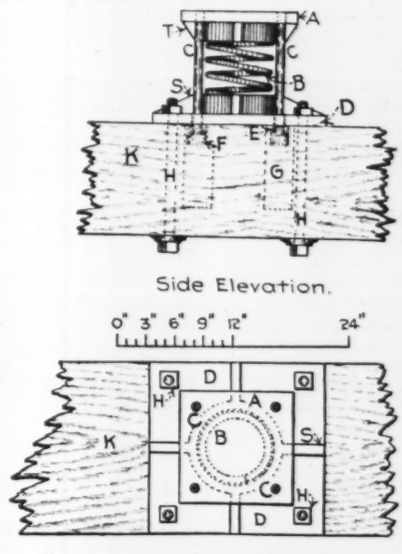

Fig. 2. Spring Dynamometer Used on Lake Superior.

Three of these dynamometers were mounted, with their pressure plates vertical, one above the other, on the sharp apex of the South Pier, Duluth Canal, at the following heights: Dynamometer "C," with its center 1.7 ft. below water level; "F," 2 ft. above water level; and "A," 5.3 ft. above water level. The water in front was about 24 ft. deep. Waves broke just in front of the dynamometers, and there was no concentration of wave energy due to reflection. The following are a few typical maximum readings of the dynamometers in lbs. per sq. ft.:

Dynamometer "C"	250	370	1,630
Dynamometer "F"	1,150	1,075	1,930
Dynamometer "A"	1,030	780	2,050

Maximum readings of 2,370, 2,195 and 1,370 lbs. per sq. ft. were given by dynamometers of the same type, located 5.3 ft., 10.9 ft. and 14.5 ft. above still-water level, respectively.

Table X. gives further data of the readings of these dynamometers, corresponding to waves of given height, length and velocity.

The variation of the maximum dynamometer readings is shown in Fig. 3. All readings connected by dotted lines were taken on the same date, but it is probable that the connected readings were not made by the same wave.

One dynamometer, "E," was fastened to a horizontal timber projecting several feet beyond the end of the south pier, Superior Entry, its pressure plate being vertical and 2 ft. above water level. The conditions were the same as for dynamometers "C," "F" and "A," and the records of results are given in Table X.

At the upper entrance of the Portage Canals, Mich., there are two timber-crib breakwaters making an angle of 90° with one another, the entrance channel being between their outer extremities. Dynamometers "K" and "L" were set on the west breakwater, facing northwest; "K" being at the junction of the sloping face with the deck; "L" being on the deck, 15 ft. to the rear and 10 ft. to the side of "K." Both were about 7½ ft. above the water level. The maximum reading on "K" was 2,525 lbs. per sq. ft. The maximum reading on "L" was 1,960 lbs., and this was the only day on which the maximum reading on "L" exceeded the maximum on "K." Table XI. gives other records. The depth in front of these dynamometers was 29 ft. In very severe storms the crests of the waves were higher than the deck of the breakwaters.

It is interesting to note that, in spite of popular belief to the contrary, the force exerted by waves against dynamometer "K," near the front wall of the breakwater, was, on the average, about 30% greater than that exerted against dynamometer "L," near the back of the wall.

It is interesting to note how nearly alike are the maximum dynamometer readings at each of the most exposed localities. For example, at Duluth Canal, Superior Entry, Portage Canal, and Black Rock, the maximum readings are 2,370, 2,255, 2,525 and 2,055 respectively.

To ascertain the relative values of horizontal and vertical action, dynamometers "M" and "O" were mounted on the vertical face of a concrete sea wall near the Duluth Ship Canal. "O" was fastened to the wall with its face vertical, its center 2¼ ft. above water level, and the face of the pressure plate 1 ft. away from the face of the wall. A few feet distant "M" was set, face downward, with the center of pressure plate 10 ins. from the face of the wall, and 2 ft. above water level. The depth of water directly in front of the dynamometers was 5 ft., and 200 ft. lakeward it was 6½ ft. The following are some of the maximum readings (lbs. per sq. ft.) at different dates, Aug. to Dec., 1902:

"O"	0	2,430	1,160	2,490
"M"	1,100	2,725	790	1,100

In 5 out of 10 storms dynamometer "O" gave no reading, while dynamometer "M" gave a reading every time. However, if the pressure plate of "O" had been flush with the wall, there would have been no back pressure, and pressure records might have been made. Spring dynamometers like these do not appear well adapted for use when fastened to a wall having large area, where the form of the wall tends to prevent water in rear of pressure plate from moving freely away. Readings may be too small, due to counterpressure, but they cannot be too large. With the strong springs used it is not probable that the momentum of the moving parts could materially increase the reading.

In order to secure an instrument which would register both static and dynamic wave pressures, and could be read for each individual wave, the writer in 1902 devised the type of diaphragm dynamometer shown in Fig. 4. In this instrument the cast-iron flanged cylinder A is covered with a diaphragm of ¼-in. rubber belting, R, held in place by the cast-iron ring E, and bolts F. Its exposed face is exactly 1 sq. ft. in area. To prevent leakage, the faces of the flange and ring are machine-planed. A set-screw, S, is provided to let out the air when the cylinder is being filled. A ¾-in. pipe, P, leads to a small tank, T. A ½-in. pipe with a downward inclination leads the gage, G, which is of the modified Bourdon type, and registers pressures up to 30 lbs. per sq. in. The gage was 14 to 19 ft. above the diaphragm, and 6 to 12 ft. from it horizontally. To lessen the momentum of the moving parts, the interior of the gage, G, was filled with glycerin. After filling the dynamometer and pipes with water, the valve C was closed, and valve B opened. Any pressure upon the dynamometer was then registered upon the gage in pounds per square inch.

The instruments were rated by submergence in still water.

In testing the dynamometer it was found that it would register correctly a pressure applied and removed in less than a second; and, as the period of the ordinary storm wave is 6 to 9 secs., it was believed that actual wave pressures would be correctly registered.

One of the dynamometers was set with its diaphragm horizontal and 2 ft. below still water level, and by means of this instrument there were obtained 70 readings of static pressures due to passing waves.

More than 1,000 readings for individual waves were taken with two diaphragm dynamometers, No. 1 having the center of the diaphragm 3.5 ft. above water level, and No. 2 1 ft. below water level. The most important of these readings are given in Table XII. Unfortunately, only two storms, neither severe, occurred after the placing of these dynamometers on the outer end of South Pier, Duluth Canal.

COMPARISON OF THEORETICAL AND ACTUAL WAVE FORCE.—When a wave breaks, the mass of water forming the top of the wave moves forward with a horizontal velocity at least as great as, and usually greater than, the velocity of wave propagation. The writer has

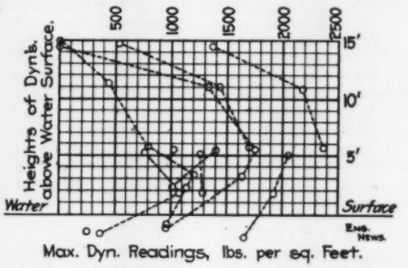

Fig. 3. Diagram Showing Wave Pressures, Lake Superior.

been unable to determine by observation the ratio of this horizontal velocity to that of wave propagation, but from careful investigation, it is believed that the maximum possible horizontal velocity is equal to the velocity of wave propagation plus the orbital velocity of a surface particle at the crest of the wave. When the breaking wave is of considerable size, a large mass of water impinges with nearly constant velocity for an appreciable length of time, therefore it would seem that the ordinary hydrodynamic formulae for the pressure of a current on a plane surface normal to the direction of flow should apply, approximately at least.

Where the direction of flow is normal to the plate, the common formula is:

$$P = f \frac{w}{2g} A v^2 \qquad (11)$$

P = normal pressure in pounds.
w = weight of cubic foot of water in pounds.
g = acceleration of gravity = 32.2.
A = area of plate in square feet.
v = velocity of current in feet per second.
f = a constant determined experimentally, but with a limiting value of 2.

For a fixed plane in a moving current of water Mariotte found $f = 1.25$, and Dubuat found $f = 1.856$.

Making the reasonable assumption that the maximum dynamometer reading on any date corresponds to the maximum observed wave dimensions on the same date, we can test formula (11). Tables IX. to XII. give the observed and computed pressures, and the calculated maximum values of f, which are given in the column headed f_m, deduced from the highest single reading in any case. In equation (11), $\frac{w}{2g} A = 1$ for all the dynamometers, w = 62.4 lbs. in fresh water, and 64.4 lbs. in salt water. For v there has been substituted v + v″; v being the velocity of wave propagation observed, and v″ the calculated orbital velocity. The maximum dynamometer readings in the eighth column are the means of the entire number of observations indicated in the last column, and the values of f given in the sixth column correspond to these means. It will be noted in Tables that the mean value of f is nearly the same as that determined experimentally in running water by Mariotte, but is considerably smaller than the values determined by Rayleigh, Joessel, Froude and Dubuat, which are probably more nearly correct. It will be noted that in several cases f_m exceeded 1.8, and in one case equalled 2. It will also be noted that the mean value of f for the diaphragm dynamometer given in Table XII. is almost identical with that deduced for the spring dynamometers in Tables IX., X. and XI.

As a result of what precedes, it may be stated that the greatest damage to marine work is usually caused by a large mass of water rushing against it for an appreciable length of time, causing a pressure which appears to conform to that indicated by well-established hydrodynamic laws for a current flowing against a submerged plane.

It will now be further shown: (1) That the impact of a wave does not resemble that of a solid body; (2) that a mass of water in air projected with a certain velocity against a plane surface can produce no greater pressure than would be caused by the steady flow against this surface of a jet of equal cross-section having the same velocity and striking at the same angle; and (3) that the pressures indicated by spring dynamometers, as usually constructed, are due to dynamic action only.

To determine the nature of wave impact the following experiments were carried out: A fire hose with a 13-16-in. nozzle was coupled to a city hydrant where the ordinary pressure was 105 lbs. per sq. in., and could be regulated by a valve. The velocity was ascertained by discharging the jet into a barrel for a definite period of 20 to 30 seconds. The pressure due to the jet of water was measured by a spring dynamometer having a circular pressure disk of exactly ½ sq. in. in area. Some of the observed and computed pressures are given in the following table, the formula for computing being:

$$p = \frac{w}{144} \frac{v^2}{2g} = \frac{62.4}{144} \frac{v^2}{2g} \qquad (12)$$

TABLE XIII.

v =	15.5	27.0	33.2	42.3	73.0	82.8
Theoretical pressure (p)	1.6	4.9	7.4	12.0	33.7	45.9
Observed pressure (p′)	1.4	4.4	6.8	12.2	34.2	40.2
Ratio of p′ to p	0.88	0.90	0.92	1.02	0.96	0.88

The first three tests were made with a light-spring dynamometer; the last three with a heavy-spring dynamometer.

To determine the pressure due to sudden impact, a shutter was used to intercept the jet, the shutter cutting across the stream like the blow of an ax. The actual velocity of the shutter probably did not exceed the velocity of the jet, and would therefore give the jet an oblique front. In not a single instance did the sudden impact of the interrupted jet give a reading in excess of that due to steady and continuous flow at the same velocity.

Other experiments were made by quickly upturning a vessel of water, containing 2½ to 3½ gallons, so as to allow a mass of water to fall 5 to 9 ft. on the spring dynamometer. One hundred and fifty tests were made, yet in no case did the observed pressure exceed 88% of the calculated static pressure due to a head equal to a height of fall of 5 ft., and 92% due to one of 9 ft.

For purposes of comparison water was dropped upon a small (1-in.) diaphragm dynamometer. In 6 out of 91 observations the observed reading equalled the theoretical pressure due to a static head equal to the height of fall, the mean of all being 81% of the theoretical pressure. In just one instance the reading exceeded the theoretical by 17%, and in this instance the gage needle moved with such great rapidity that its momentum may have carried it beyond its true position.

To show the effect of impact of a solid body upon the same spring dynamometers, pieces of iron, weighing from 1 to 16 oz., were dropped from a height of 1 ft., with the following results:

Weight dropped, lbs.	0.0625	0.125	0.25	0.50	1.00
Weight registered, "A," lbs.	1.36	1.82	2.60	3.66	...
Weight registered, "B," lbs.	4.12	5.81	8.31	12.0	17.25

The strength of spring "A" was 1.4 lbs. per in. of compression; spring "B," 12.1 lbs. per in. of compression. From these tests it would appear that the impact of a mass of water exerts a true pressure, and does not produce at all the same effect as that of a solid body.

An inspection of this table shows that, for equal force

Table IX.—North Beach, St. Augustine, Fla., 1890-91.

Maximum wave dimensions. Height. Ft.	Length. Ft.	Velocity f. s.	V″ (computed). f. s.	(V + V″)²	$f = \frac{2gP}{w(v+v'')^2}$	f_m	Maximum dynamometer readings (pressure per sq. ft.) Observed. Lbs.	Computed. Lbs.	Number of observations.
2.0	46	8.4	2.9	127.7	1.16	....	148	168	1
2.5	60	9.4	3.2	158.8	1.45	1.69	230	209	12
2.75	70	10.3	3.5	190.4	1.41	1.69	269	249	5
3.0	75	11.7	3.7	237.2	1.36	1.80	322	310	20
3.5	78	12.0	4.0	256.0	1.22	1.48	313	335	8
4.0	82	12.2	4.1	265.7	1.53	2.00	406	348	20
4.5	90	14.0	4.9	357.2	1.27	1.81	452	469	15
5.0	120	15.2	5.3	420.2	1.11	1.80	467	550	16
5.5	130	16.7	5.7	501.8	1.10	1.64	550	657	3
6.0	150	18.2	6.2	595.4	1.12	1.44	667	780	7
	Total								107
	Mean				1.32	1.71			

Table X.—Duluth Canal and Superior Entry, Lake Superior, 1901-02.

12.0	150	24.2	7.3	992.2	1.20	....	1,150 (F)	1,259	1
12.0	130	24.2	7.9	1,030.4	1.20	....	1,190 (E)	1,307	1
16.0	250	33.2	10.6	1,918.4	1.22	....	2,255 (E)	2,432	1
10.0	150	23.7	6.0	882.1	1.41	....	1,210 (D)	1,118	1
13.0	150	29.6	9.8	1,552.4	1.07	....	1,615 (D)	1,968	1
14.0	150	27.2	9.6	1,354.2	1.23	....	1,605 (A)	1,718	1
16.0	200	30.0	10.3	1,624.1	1.12	....	1,755 (C)	2,059	1
18.0	250	32.0	11.5	1,892.2	1.29	....	2,370 (C)	2,400	1
16.0	210	31.0	10.4	1,714.0	1.02	....	1,700 (C)	2,174	1
	Total								9
	Mean				1.20				

Table XI.—Upper Entrance, Portage Canals, Lake Superior, 1902.
(Center of dynamometer set 7½ ft. above water level.)

13.5	200	30.0	8.5	1,482.2	1.36	....	1,960 (L)	1,879	1
....	....	..	....	1,444.0	1.31	....	1,880 (K)	1,831	1
....	....	....	....	1,436.4	1.38	....	1,920 (L)	1,824	1
	Total								3
	Mean				1.35				

Mean value of f for all observations = 1.31.

Table XII.—Outer End of South Pier, Duluth Canal, 1902.

4.8	50	16.0	4.7	428.5	.99	....	410*	542	....
5.0	50	16.0	4.9	436.8	1.26	....	531*	553	....
5.0	70	19.0	4.2	538.2	1.30	....	677*	681	....
6.0	90	21.3	4.6	670.8	1.33	....	864*	850	....
10.0	150	25.0	6.3	979.7	1.28	....	1,210†	1,241	....
13.0	200	30.0	8.5	1,482.2	1.14	....	1,642†	1,877	....
	Mean				1.22				

* Dynamometer No. 2.
† Dynamometer No. 1.

of impact, the stronger spring registers pressures more than three times greater than those given by the weaker spring, while in the case both of the continuous and of the interrupted jets of water, the resulting pressures were practically unaffected by the strength of spring.

It apparently follows from all the above experiments that the most destructive waves act for an *appreciable period*, the pressures which they exert can properly be measured by suitably constructed dynamometers.

STORM WAVES IN THE DULUTH CANAL, 1901.
(Wind velocity 46½ miles per hour. Water and spray thrown to a height of 40 ft.)

OBSERVED EFFECTS OF WAVE ACTION.—[Table XIV. contains a summary of the observed effects of wave action in damaging breakwaters, piers, etc. Stone or concrete blocks of known weight have been moved en mass, either sliding or overturning, in examples Nos. 1, 2, 3, 4, 8, 9, 10, 12. Coefficients of sliding friction of 0.6 to 0.85 were assumed, and deductions were made in the weight of the blocks due to water displaced. In cases Nos. 5 and 6, timber cribs were overturned by the wave force, and the pressure required to overturn them was estimated. For the column giving computed dynamometer pressures, equation (11) was used, assuming $f = 2$, and assuming the probable wave dimensions given in the last two columns for computing the velocity.

The most wonderful action of a wave upon a large area is given in example No. 1, as recorded by Stevenson, a mass of concrete weighing 1,350 long tons, or 1,510 short tons, being slid off its masonry foundation by the pounding of the waves, without injuring the foundation. Major Gaillard has assumed the frictional resistance to be 0.75 of the submerged weight of this mass.

In example No. 5 a rock-filled crib was overturned, requiring a force of 1,100 lbs. per sq. ft. over an area of 2,250 sq. ft., as computed by the officer in charge, assuming the center of pressure of wave impact to be at the center of the exposed face of the breakwater—although it probably was higher. Example No. 6, at Milwaukee, was a crib, likewise overturned.

Example No. 7 was the breaking of the 12 × 12-in. timber deck of the Buffalo Breakwater, due to a mass of water falling vertically upon it, after the breaking of a wave. The unit-breaking strength of the timber was assumed as being 6,000 lbs. per sq. in. In example No. 8, the resistance to sliding of the concrete mass was assumed to be 0.75 of the 122 tons submerged weight.

Example No. 11 was the breaking of wooden ties and decking of a breakwater by a mass of falling water.

In all cases it must be remembered that the actual pressures on the structures were doubtless considerably in excess of the pressure estimated as necessary to cause destruction.—Ed.]

Within certain limits, the smaller the area the greater will be the force exerted upon it by water falling upon or by waves projected against it.

CONDITIONS MODIFYING EFFECTS OF WAVE ACTION.—No wave can exist in water of depth less than the height of the wave. Hence it follows that in many cases the most violent storms do not produce the greatest effect upon engineering structures, for the large waves of such a storm will often break before reaching the structure, whereas the smaller waves of a less violent storm may pass unbroken and reach the structure.

The slope and configuration of the bottom near a structure influence the action of waves, tending in some cases to concentrate, in other cases to reduce wave action.

In northern latitudes huge masses of ice may form on structures, resulting from the freezing of water and spray thrown upon the structure by breaking waves. This ice imposes an additional and eccentric load upon the structure, raises the center of gravity, and increases the area exposed to wave action. See Figs. 6 and 7.

In March, 1902, the west breakwater at the upper entrance of Portage Lake canals, Lake Superior, was covered by a solid mass of ice about 2,000 ft. long, 30 to 40 ft. wide, and 20 to 25 ft. above the deck. Clear ice in cold weather has a tensile strength of about 312 lbs. per sq. in., as shown by numerous tests in the laboratory of the U. S. Engineer office at Duluth, Minn. As long as the ice mass is firm and continuous, its action is similar to that of a monolithic concrete structure. When it begins to melt, and is partly washed away by the waves, it frequently becomes a menace to the work. During the break-up of ice in the spring, damage is sometimes caused by blocks of ice hurled by the waves against structures.

BLOCKS OF ICE THROWN UP BY WAVES AT MARQUETTE, MICH.

DESIGN OF BREAKWATERS.—Breakwaters, as usually constructed, are of three general types: (1) In which the exposed face is vertical; (2) partly vertical and partly inclined; and (3) inclined face.

The vertical face breakwater is at times subjected to very heavy wave impact, although most of the waves which would attack it are reflected, and, meeting those advancing produce by wave interference a chaotic movement of the water for a considerable distance out from the work. Some of the oncoming waves are neutralized and a small number augmented, so that the vertical wall occasionally receives an impact greater than would be the case if no reflection occurred.

In many cases a combination of sloping and vertical faces is used in the same breakwater. The superstructure in such cases is sometimes subjected to heavier impact than that of types (1) or (3), owing to the fact that the waves break upon the slope and act with concentrated effect upon the superstructure.

By some writers it has been claimed that the mass of water which passes over the sloping face of a breakwater and falls in the rear causes swells or waves of size sufficient to incommodate shipping. This may be the case where the top of the breakwater is low, but is not true where it is given a reasonable height. For, example, the sloping-face breakwater at the Portage Canals, Lake Superior, is 7 ft. above ordinary storm level, and is 19 ft. wide on top. The inclined face has a slope of 1 to 1, which extends 4 ft. below storm level of the lake. The rear wall is vertical, as is also the front wall below the slope. During severe storms waves from 12 to 15 ft. in height assail this breakwater, and at times immense masses of water pass over it, falling at some distance in the rear, but producing no waves or swells of appreciable size. The action of this breakwater in reflecting waves is very marked, a perfect chaos of waves extending lakeward for 1,000 ft.

From evidence available it would appear: (a) That a sloping-face breakwater with vertical back wall, located in water less than 30 ft. deep, will afford sufficient protection to the area in the rear, provided its height above water during storms is not less than two-thirds the height of the greatest waves that assail it; (b) that, unless the height and top width of the breakwater are unduly great, so much water will pass over the top during severe storms that it would wash completely over the decks of vessels alongside the breakwater; (c) that both the vertical and sloping-face breakwaters will reflect waves, provided the height is sufficient, even although the angle of slope with the horizontal is as small as 23 degrees.

Mr. J. H. Darling, U. S. Assistant Engineer, in the report of the Chief Engineer for 1901, has shown that, against overturning, the volumes of two cubes of different densities, but of equal stability, are inversely proportional to the cubes of their densities. Mr. Darling has also shown that the force required to overturn cubes of the same size, but of different materials, is directly proportional to the densities of the materials in air, or to their net weights if immersed in water. For different sizes of cubes of the same material, the force per unit of surface required to overturn them is proportional to their linear dimensions. All of the foregoing relations hold true of similar polyhedrons similarly placed with respect to the force acting upon them, and also approximately to rip-rap where there is a general similarity among pieces of different sizes. Blocks deposited in a pile mutually support and protect one an-

Table XIV.

No.	Locality.	Weight moved, tons.	Area exposed, sq. ft.	Pressure lbs. per sq. ft. to move.	Maximum Dynamometer —pressure lbs. per sq. ft.— Observed.	Computed.	Estimated Wave —dimensions, ft.— Height.	Length.
1	Wick, Scotland	1,510	546	2,015	....	6,340	42	500
2	North Beach, Florida	10.8	60	200	633	....	....	....
3	"	2.3	12.5	200	533	....	....	....
4	"	1.8	11.4	132	425	....	....	....
5	Milwaukee, Wis.	....	2,250	1,100	....	2,436	12	200
6	"	....	1,325	797	540	2,409	13	200
7	Buffalo, N. Y.	....	50	1,145	....	1,675	10	100
8	"	208	252	726	....	1,675	10	100
9	"	19	36	415	....	1,675	10	100
10	Marquette, Mich.	4.5	10	318	2,055	....	....	....
11	"	....	72	993	2,055	....	....	....
12	Duluth, Minn.	6.9	19	714	1,630	....	....	....

other to some extent. The relations for isolated pieces, above given, may be taken as applying approximately to rip-rap loosely piled, and less accurately to pieces closely piled.

The volumes of two cubes of different densities, but of equal stability, again sliding, are inversely proportional to the cubes of their densities. When the coefficient of friction is less than 1, a cube will slide rather than overturn. The coefficient of sliding friction varies from 0.25 to 1.11 or more, and is practically the same in water as in air.

Relative sizes of cubes of different densities to give equal resistance against overturning or sliding:

Weight per cu. ft. in air.....	180	170	160	150	140	130
Relative sizes of cubes in air.	1.0	1.2	1.4	1.7	2.1	2.6
Relative sizes in water.....	1.0	1.3	1.7	2.4	3.4	5.3

It is obvious that from motives of economy and efficiency it is inadvisable to expend labor in carefully dressing the beds of masonry to be used in harbor construction, since polished stone has a coefficient of friction 25 per cent. less than roughly dressed stone.

In carrying on concrete work in exposed situations the mistake is frequently made of attempting to use molds of insufficient strength. Sudden storms are apt to interrupt the work, and molds should be strong enough to resist the waves, and tight enough to prevent the concrete from being washed out before it has set. It was found at North Beach, Fla., that Portland cement concrete could withstand wave action 65 hours after being placed, even with a wave pressure of 509 lbs. per sq. ft. The tensile strength of the concrete at the time was about 29 lbs. per sq. in. At the same place concrete 48 hours old was washed slightly on its upper surface during the storm. These results accord with investigations made on the Cherbourg breakwater, which showed that the cohesive strength required in mortar to withstand the battering of waves should be at least 22¾ lbs. per sq. in.

The kind of timber most commonly used on the Great Lakes is white pine. It possesses the disadvantage of being very light, requiring a greater amount of ballast than heavier wood requires. Moreover, it does not become "water logged." White pine timbers forming the old crib at the Duluth Canal, when torn apart for removal, floated after 30 years submergence.

Owing to the buoyancy of the timber, the rock-filled timber cribs on the Great Lakes have a net weight under water of about 50 lbs. per cu. ft. of displacement. The integrity of the cribs depends largely upon the decking remaining in place, for when the decking is displaced the waves wash the ballast out and may destroy the crib to a depth of 12 ft. below the water surface. It should be remembered that a concentrated wave pressure upon a limited area may, by hydrostatic pressure develop its force upon an area many times greater. Spikes are frequently drawn by wave pressure on timbers; lag screws, or bolts with nuts and washers are, therefore, preferable, and in exposed places the threads of the screws should be upset after the nuts are in place.

Finally, it should be remembered that injury to marine constructions occurs oftener while work is in progress than after completion.

A TRAFFIC CENSUS ON ASPHALT AND GRANITE PAVEMENTS IN NEW YORK CITY.

In a paper read by Captain (now General) Francis V. Greene before the American Society of Civil Engineers, Dec. 16, 1885, the results were given of certain observations made with the view of determining the actual and comparative traffic on the streets of certain cities, and whether the pavements on these streets properly fulfill the object for which they are designed. Since that time no further observations of a similar nature have been made in New York, with the exception of a count of the number of vehicles passing per day at four points on Fifth Ave. in 1896, before that street was paved with asphalt. On this account the Barber Asphalt Paving Co. in the months of November and December, 1904, made an accurate determination of the traffic on some of the most important streets in New York, paved with both asphalt and granite block, with two objects in view: First, to determine how much the traffic had increased in 19 years; and, second, how large a tonnage asphalt pavement, such as that on Fifth Ave., which has been extremely successful, might be able to carry without disintegration. A summary of the results are presented in tables I and II.

It should be stated that in estimating the weight of loads, the same assumptions were made as those made by General Greene in 1885.

By reference to the tables given by Capt. Greene, it appears that on 36 streets in ten cities the heaviest traffic was found on Broadway, near Pine, in New York, 273 tons per lineal foot of width of street, while at Fifth Ave., opposite the Worth Monument, it was only 94 tons. From the data recently collected, it seems that none of the streets on which a count was made in 1904 carried a traffic less than that of Broadway in 1885, with the exception of one cross-town street, 34th St.

The most important comparison is in regard to the tonnage per lineal foot carried by Fifth Ave. in 1885 and in 1904, after a period of 19 years. In 1904 it was found to be 481 tons per lineal foot of width, where in 1885 it was only 94, showing that the traffic has increased five-fold in that period. The vehicles in 1885 were 5,460; in 1896, 6,300, before the street was paved with asphalt; and in 1904, 11,787. The average weight per vehicle for 1886 on Fifth Ave. was 0.68 tons; in 1904 it was 1.64 tons. On Broadway in 1886 it was 1.39 tons and in 1904, 1.97 tons. From this the conclusion is drawn that a very much larger number of heavy teams use Fifth Ave. at the present time than 20 years ago, while there has been a decided increase in the average weight of the vehicles on the other streets.

An examination of the data in regard to streets paved with granite blocks shows that these streets, either owing to the fact that they are paved with stone, and consequently, have not a smooth surface, or are not main arteries of travel, do not begin to carry the heavy traffic that the streets paved with asphalt do.

As a whole, the results are very surprising as regards the increase in traffic in 20 years, and as showing that an asphalt pavement, when well constructed on a concrete base, is able to carry a traffic which very nearly equals the latest data which are available as to that on heavy traffic streets in London and Paris, as shown in quotations by Capt. Greene from a paper by Mr. Howarth before the Institution of Civil Engineers in 1879, from which it appears that the heaviest traffic was on Grace Church St. in London, which amounted to 422 tons per foot width for the entire 24 hours, as compared with 482 tons on Fifth Ave. during 11 hours; in the former case, 12,148 vehicles representing the traffic, and in the latter 11,787 vehicles.

The data as a whole point to the fact that a well-constructed asphalt pavement is quite suited to sustain the heaviest traffic with which we are acquainted.

For the foregoing information we are indebted to Mr. Clifford Richardson, Director New York Testing Laboratory, Long Island City, N. Y.

STONE AND CONCRETE RETAINING WALLS FOR TRACK ELEVATION IN CHICAGO.

In the joint track elevation work of the Chicago & Northwestern Ry. and the Pittsburg, Cincinnati, Chicago & St. Louis Ry. at Chicago, an interesting feature is that on the north side of the right of way is a stone masonry retaining wall

TABLE I.—TRAFFIC RECORD TAKEN ON STREETS PAVED WITH ASPHALT IN NEW YORK CITY.

Fourth Street, from Wooster Street to West Broadway. Width of street = 32 ft. Averaged for Nov. 30 and Dec. 1, 1904.

Between the hours of.....	7 and 8	8 and 9	9 and 10	10 and 11	11 and 12	12 and 1	1 and 2	2 and 3	3 and 4	4 and 5	5 and 6	Total.
Tonnage per hour.....	487.67	862.48	953.32	1,006.97	1,010.61	723.67	745.42	827.23	854.02	855.00	927.83	9,254.22
Tonnage per lineal foot per hour.....	15.24	26.95	29.79	31.47	31.58	22.61	23.29	25.85	26.69	26.72	28.99	289.18

Average tonnage per hour = 841.35. Average tonnage per lineal foot of width per hour = 26.27.

Eighth Avenue, from 35th to 36th Streets. Width of street less car tracks = 44 ft. Averaged for Nov. 22 and 23, 1904.

Between the hours of.....	7 and 8	8 and 9	9 and 10	10 and 11	11 and 12	12 and 1	1 and 2	2 and 3	3 and 4	4 and 5	5 and 6	Total.
Tonnage per hour.....	1,010.07	1,067.27	1,260.92	1,226.97	1,244.45	1,229.05	1,197.25	1,140.65	1,269.32	1,326.45	1,052.12	13,024.52
Tonnage per lineal foot per hour.....	22.96	24.26	28.66	27.89	28.28	27.93	27.21	25.92	28.85	30.15	23.91	296.02

Average tonnage per hour = 1,184.05. Average tonnage per lineal foot of width per hour = 26.91.

Thirty-fourth Street, from Broadway to Seventh Avenue. Width of street less car tracks = 24.4 ft. Averaged for Dec. 7 and 8, 1904.

Between the hours of.....	7 and 8	8 and 9	9 and 10	10 and 11	11 and 12	12 and 1	1 and 2	2 and 3	3 and 4	4 and 5	5 and 6	Total.
Tonnage per hour.....	161.25	257.30	185.08	211.53	216.38	173.10	127.25	221.50	230.70	238.28	154.23	2,176.60
Tonnage per lineal foot per hour.....	6.61	10.55	7.59	8.67	8.67	7.09	5.22	9.07	9.46	9.77	6.32	89.22

Average tonnage per hour = 197.86. Average tonnage per lineal foot of width per hour = 8.10.

First Avenue, from 26th to 27th Streets. Width of street less car tracks = 44 ft. Averaged for Nov. 25 and 26, 1904.

Between the hours of.....	7 and 8	8 and 9	9 and 10	10 and 11	11 and 12	12 and 1	1 and 2	2 and 3	3 and 4	4 and 5	5 and 6	Total.
Tonnage per hour.....	1,300.93	1,798.51	2,156.85	1,950.25	1,443.70	1,686.07	1,991.91	1,990.02	1,473.35	1,778.97	1,283.20	19,253.76
Tonnage per lineal foot per hour.....	28.56	40.87	49.02	44.32	41.90	38.42	45.27	45.24	33.49	49.43	29.16	435.58

Average tonnage per hour = 1,750.34. Average tonnage per lineal foot of width per hour = 39.78.

Fifth Avenue, from 33d to 34th Streets. Width of street = 40 ft. Averaged for Nov. 14, 15, and 16, 1904.

Between the hours of.....	7 and 8	8 and 9	9 and 10	10 and 11	11 and 12	12 and 1	1 and 2	2 and 3	3 and 4	4 and 5	5 and 6	Total.
Tonnage per hour.....	833.71	941.35	1,337.07	1,889.53	2,236.15	1,966.96	1,652.06	1,833.70	2,433.88	2,233.65	1,916.41	19,274.47
Tonnage per lineal foot per hour.....	20.83	23.54	33.43	47.24	55.90	49.17	41.30	45.84	60.85	55.84	47.91	.....

Average tonnage per hour = 1,752.20. Average tonnage per lineal foot of width per hour = 34.80.

Broadway, from 18th to 19th Streets. Width of street less car tracks = 25 ft. Averaged for Nov. 17 and 18, 1904.

Between the hours of.....	7 and 8	8 and 9	9 and 10	10 and 11	11 and 12	12 and 1	1 and 2	2 and 3	3 and 4	4 and 5	5 and 6	Total.
Tonnage per hour.....	272.15	539.68	582.17	859.63	875.85	1,013.12	567.00	625.70	704.40	803.30	648.90	7,491.70
Tonnage per lineal foot per hour.....	10.88	21.58	23.28	34.38	35.02	40.52	22.68	25.03	28.17	32.13	25.96	299.63

Average tonnage per hour = 681.06. Average tonnage per lineal foot of width per hour = 27.23.

SUMMARY.

Street.	Tonnage per 11 hours, from 7 a.m. to 6 p.m.	Average tonnage per lineal foot of width for 11 hours.	Average tonnage per hour.	Average number vehicles per 11 hours.	Average tons per vehicle.
Fourth Street, from Wooster to West Broadway.....	9,254.22	298.18	841.35	3,394	2.73
Eighth Avenue, from 35th to 36th Streets.....	13,024.52	296.02	1,184.05	5,720	2.28
Thirty-fourth Street, from Broadway to Seventh Avenue.....	2,176.60	89.22	197.86	1,072	2.03
First Avenue, from 26th to 27th Streets.....	19,253.76	435.58	1,750.34	6,034	3.18
Fifth Avenue, from 33d to 34th Streets.....	19,274.47	481.85	1,752.20	11,787	1.64
Broadway, from 18th to 19th Streets.....	7,491.70	299.63	681.06	3,817	1.97

built by the former road, while on the south side is a concrete retaining wall built by the latter road. This joint work extends along the line of Kinzie St. for several blocks east from Western Ave. The railway properties are adjacent and include main and side tracks, industry tracks, etc., with constant train and switching movements.

The Chicago & Northwestern Ry. used a steam shovel in excavating the trench, and concrete-making trains (each with cars of cement, sand and crushed stone, and a concrete mixing machine) for putting in the foundation course. The stone blocks for the wall (from the railway company's own quarry) were delivered on flat and gondola cars on a track alongside of the foundation, and were handled and set in place by derrick cars running on rails laid upon the concrete. The Pittsburg, Cincinnati, Chicago & St. Louis Ry. has used concrete-making trains, similar to the above, for all its work. Figs. 1 and 2 show the

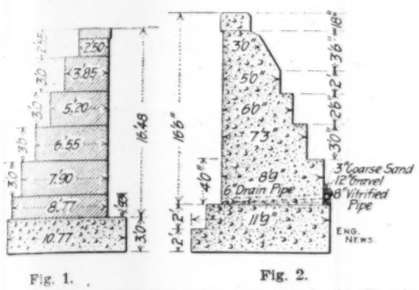

Fig. 1. Fig. 2.

Fig. 1. Stone Retaining Wall on Kinzie St. Track Elevation at Chicago; Chicago & Northwestern Ry.

Fig. 2. Concrete Retaining Wall on Kinzie St. Track Elevation at Chicago; Pittsburg, Cincinnati, Chicago & St. Louis Ry.

sections of the two walls, and it is stated that the masonry wall has been built as cheaply and readily as a concrete wall. The walls are now partly built, and next spring they will be completed and the filling will be put in. The concrete foundations for the bridge abutment walls (transverse to the tracks) were put in with little disturbance of the traffic. Shallow trenches were first excavated, spanned by timber stringers for the tracks, and the deep trenches were then excavated, in which the foundations were built. The space above each foundation was then filled in up to the tracks. When the work is to be continued, certain tracks will be removed, the foundations uncovered, and the abutments built up ready for the plate girder bridges over the streets.

This piece of track elevation is an important part of the general scheme of eliminating grade crossings of streets in Chicago. It includes the Chicago & Northwestern Ry. main line (Galena Division) from Sacramento Ave. (the present eastern end of its track elevation) east to Ada St.; and the Pittsburg, Cincinnati, Chicago & St. Louis Ry. from the end of its track elevation at Campbell Ave. east to Ada St. The Chicago, Milwaukee & St. Paul Ry. has trackage rights over the latter railway east from Western Ave. to the Union Station, and will therefore elevate its own tracks from Western Ave. north to its present track elevation at Kedzie St. There will still be a grade crossing of the elevated tracks of the Chicago & Northwestern Ry. and Chicago, Milwaukee & St. Paul Ry. at Western Ave., but this will be protected by an interlocking plant, so that the present stops of all trains at this crossing will be eliminated.

THE MAIN STREET CROSSING OF THE CANADIAN PACIFIC RY., AT WINNIPEG, MAN.

The drawings on pages 196 and 197, illustrate some interesting reinforced concrete work which forms a part of the extensive terminal improvements now being prosecuted by the Canadian Pacific Ry. at Winnipeg, Manitoba. The full list of improvement works comprises new freight yards of large size, a passenger terminal, hotel, car and locomotive shops, and a considerable amount of grade crossing improvement. The most important item of work in the last mentioned task is the abolition of the former grade crossing over Main St., the principal business thoroughfare of the city.

The Canadian Pacific Ry. main line passes through the center of the city, the right of way being a grant made to the company as a concession for the lines to enter the city. At the time this grant was made and until some four years ago this bisecting of the city caused but little inconvenience, but with the rapid expansion of the city and the Canadian west the company found it imperative that they should have more tracks at this point. Application was made to the Railway Committee of the privy council for the right to lay more tracks, but the citizens objected on the ground that such construction would practically separate the two parts of the city, to the detriment of land values in the northern portion. It was finally agreed that a subway should be built, with sufficient accommodation to meet the requirements of all parties concerned for all time to come. A year or more was lost in dickering between the city and the company, but the project as finally settled upon was for a concrete subway, 100 ft. in width, carrying eight tracks, and giving a headroom of 14 ft. 6 ins. for carriages and electric car lines. The company was to build the structure at an estimated cost of $190,000, pay all legitimate damages to adjoining property, and in return was allowed to close several cross streets in the vicinity, and was given space on Point Douglas Ave. for the additional lines of rails.

There were several difficulties to be contended with in the work, the first of which lay in the fact that on account of the low level of this portion of the city, only 13 ft. above extreme high-water mark, it was impossible to carry the excavation to the full depth required. A compromise was made and the company raised the level of its tracks 5 ft. for a distance of three-quarters of a mile, leaving the excavation at 11 ft. 11 ins. Traffic could not be delayed during construction either on the street or railway, so that it was necessary for the contractor to keep open the east 32 ft. of the street, and also to build the subway in sections. Though the traffic on Main St. is the heaviest in the city, and the whole of the freight and passenger traffic of the railway must pass at this point, despite these drawbacks, the work was carried through without delay or accident, and the finished structure has already more than proved its worth.

The general arrangement of the subway structure is shown by the plan, Fig. 1. As will be seen it occupies only a portion of the width of the street, the retaining wall on the east side falling some 30 ft. west of the building line, and only that portion of the street coming between the retaining walls is depressed. The general characteristics of the structure are shown by the elevation and section in Fig. 2. It consists of piers founded on piles and carrying groined arches, which form the platform for the railway track. The material used is reinforced concrete.

The details of the construction are shown by the half transverse section, Fig. 3, and the part longitudinal sections, Fig. 4. The concrete consisted of a mixture of one part cement, three of sand and five of broken stone, except in the footings of the retaining walls along the approaches outside of the arch structure, where the mixture used was 1-4-7. The concrete was handled rather moist, and was thoroughly tamped. The section carrying the four south tracks was built up first, there being no intermission in the laying of the concrete from the springing line of the groined arches to the top of the finished structure, the work being carried along day and night, with relay gangs. On all exposed surfaces a facing of 1-3 mortar was used 1½ ins. thick. Tops of coping, tops of beams between tracks, and surface of platforms were finished smooth in the same. The work was given 30 days in which to set before the forms and centers of the arch structure above the springing line were struck, or the structure loaded. When these were removed it was found that there was not the slightest tendency to rise at the crown as the haunches were loaded. When the forms were removed, the whole surface was given a thin coat of grout of the consistency of whitewash, to which alum, in the proportion of ½-lb. to 4 gallons, had been added.

Deekes & Deekes, of Winnipeg, executed the contract. The design was by C. N. Monsarrat, Chief Engineer of Bridges and Buildings for the Canadian Pacific Ry., and the work was carried on under the supervision of J. E. Schwitzer, Division Engineer for the company.

TABLE II.—TRAFFIC RECORD TAKEN ON STREETS PAVED WITH GRANITE BLOCK IN NEW YORK CITY.

Tenth Avenue, from 22d to 23d Streets. Width of street less car tracks = 34 ft. Averaged for Dec. 5 and 6, 1904.

Between the hours of	7 and 8	8 and 9	9 and 10	10 and 11	11 and 12	12 and 1	1 and 2	2 and 3	3 and 4	4 and 5	5 and 6	Total
Tonnage per hour	598.65	769.50	777.93	687.48	741.48	550.43	693.88	813.50	813.53	697.75	588.45	7,732.78
Tonnage per lineal foot per hour	17.61	22.64	22.88	20.22	21.81	16.19	20.41	23.93	23.93	20.53	17.31	227.45

Average tonnage per hour = 702.98. Average tonnage per lineal foot of width per hour = 20.68.

Third Street, from Mercer to Greene Streets. Width of street = 28 ft. Averaged for Dec. 2 and 3, 1904.

Between the hours of	7 and 8	8 and 9	9 and 10	10 and 11	11 and 12	12 and 1	1 and 2	2 and 3	3 and 4	4 and 5	5 and 6	Total.
Tonnage per hour	176.83	304.45	337.68	398.18	386.58	265.50	408.90	440.40	453.13	431.85	330.30	3,933.80
Tonnage per lineal foot per hour	6.32	10.88	12.06	14.22	13.81	9.48	14.60	15.73	16.18	15.42	11.80	140.50

Average tonnage per hour = 357.62. Average tonnage per lineal foot of width per hour = 12.77.

Second Avenue, from 34th to 35th Streets. Width of street less car tracks = 46 ft. Averaged for Nov. 28 and 29, 1904.

Between the hours of	7 and 8	8 and 9	9 and 10	10 and 11	11 and 12	12 and 1	1 and 2	2 and 3	3 and 4	4 and 5	5 and 6	Total.
Tonnage per hour	184.45	242.75	311.05	287.65	250.60	238.08	239.90	286.93	254.90	242.10	191.38	2,729.79
Tonnage per lineal foot per hour	40.01	5.27	6.76	6.25	5.44	5.17	5.21	6.24	5.54	5.26	4.16	59.31

Average tonnage per hour = 248.16. Average tonnage per lineal foot of width per hour = 5.39.

Broadway, from Franklin to Leonard Streets. Width of street less car tracks = 28 ft. Averaged for Nov. 19 and 21, 1904.

Between the hours of	7 and 8	8 and 9	9 and 10	10 and 11	11 and 12	12 and 1	1 and 2	2 and 3	3 and 4	4 and 5	5 and 6	Total.
Tonnage per hour	230.02	380.15	604.31	611.52	633.52	458.12	551.40	615.20	619.82	574.98	389.18	5,668.22
Tonnage per lineal foot per hour	8.22	13.57	21.58	21.84	22.62	16.36	19.69	21.98	22.13	20.53	13.90	202.42

Average tonnage per hour = 515.29. Average tonnage per lineal foot of width per hour = 18.40.

SUMMARY.

Street.	Tonnage per 11 hours, from 7 a.m. to 6 p.m.	Average tonnage per lineal foot of width for 11 hours.	Average tonnage per hour.	Average number vehicles per 11 hours.	Average tons per vehicle.
Tenth Avenue, from 22d to 23d Streets	7,732.78	227.45	702.98	2,873	2.69
Third Street, from Mercer to Greene Streets	3,933.80	140.50	357.62	1,569	2.51
Second Avenue, from 34th to 35th Streets	2,729.79	117.86	439.01	1,222	2.24
Broadway, from Franklin to Leonard Streets	5,668.22	202.42	515.29	3,259	1.74

BARYTES AND ITS PREPARATION FOR THE MARKET.

By Edwin Higgins, Jr.*

Barytes, or heavy spar, is a white mineral, crystallizing in the orthorhombic system. The crystals are generally tabular, though often fibrous, globular or laminated. It has a hardness ranging from 2.5 to 3.5 and a specific gravity of from 4.3 to 4.6. In appearance it is vitreous, inclining to resinous. The streak is white, and, though most varieties are white, it is also found in various shades of gray, blue, yellow, red or brown.

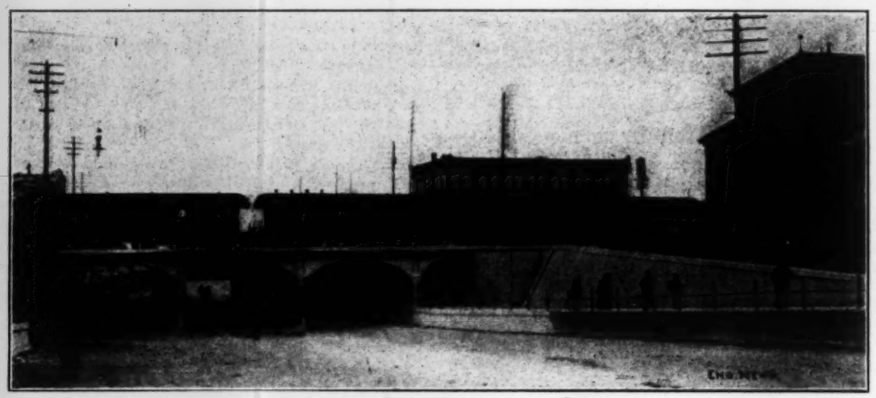

FIG. 1. VIEW OF MAIN ST. SUBWAY UNDER CANADIAN PACIFIC RY., AT WINNIPEG, MANITOBA.

Barytes occurs commonly in connection with beds or veins of metallic ores, especially of lead, copper, silver, cobalt and manganese, as part of the gangue. It is found in secondary limestones and sandstones, sometimes in veins, and also as earthy masses in beds of marl. Barytes is often found in deposits of hematite and limonite as gangue. In the knowledge of the writer an otherwise excellent vein of limonite in southwest Virginia was abandoned on account of the presence of a large per cent. of barytes.

Barytes is found in England, Europe and the United States, chiefly in Missouri, Tennessee, North Carolina and Virginia. The better grades are used in the manufacture of pigments, as a cheap substitute for, or adulterant of, white lead; and in the manufacture of reagents. It is also used in pyrotechny and as a make-weight in the manufacture of paper. Some of the lower grades are used by pork packers in the preparation of canvas for their products.

As above noted, barytes may be found with many minerals as gangue, but most of the ore mined for the market has associated with it only a small per cent. of oxide of iron, limestone, clay or sand. For the removal of these impurities washing and bleaching and in rare instances, jigging, are the only operations necessary. A dilute solution of sulphuric acid is used for the removal of the oxide of iron; and if the ore contains only stains of this, the operation is very effective and inexpensive. Chert, which will affect the color of the finished product if present in too large quantities, and limestone, which will consume sulphuric acid in the bleaching tanks, may both be removed by jigging.

The pure white ore is most abundantly mined and milled, the finished product being more valuable than that obtained from "off-color" ores.

PREPARATION OF THE ORE FOR MARKET.

The following is the usual practice followed in preparing the ore for market though the details of the several operations vary considerably at different plants. The ore is first crushed to the proper size for bleaching and then washed, after which it is ready for the bleaching tanks. The size to which the ore is reduced depends on the content and character of the iron present, varying from the size of a pea, if the iron is in the form of a scale, to pieces ½-in. in diameter, if the ore is only stained. In the former case jigging is sometimes resorted to for removing the bulk of the iron.

BLEACHING TANKS.—Circular wooden tanks, lined with sheet lead, are commonly used, though some rectangular designs are employed. They are built in various sizes, accommodating anywhere from 5 to 25 tons of ore. These tanks are usually provided, on the side and close to the bottom, with an acid discharge spout; and in the bottom with a larger discharge spout for the ore. During the operation of bleaching the ore spout is closed by means of a wooden plug, long enough to extend at least a foot above the ore when the tank is charged, to facilitate its easy removal when the ore is ready to be discharged. Some tanks have no discharge spout for ore, the bleached product being shoveled over the side of the tank into the final washer.

Each tank is provided with a steam pipe, to which is coupled a 1½-in. or 2-in. lead pipe. This lead pipe is coiled in the bottom of the tank, closed at the end, and provided with small perforations from six to eight inches apart. Steam escaping from these perforations serves to agitate and keep hot the bleaching solution. The bleaching agent is sulphuric acid, reduced to about 20° to 30° (Beaume) with water. The ore is usually charged into the tank to a depth of about 3 ft., and is followed by the proper amount of acid. Steam is now turned on and the operation of bleaching is started. The length of time required for this operation depends entirely on the content of iron in the ore and may vary from 6 to 70 or 80 hours. The usual practice is to leave the steam on continuously during the bleaching period, though good results may be obtained by cutting off the steam for half an hour at intervals of one hour. A little experience will enable one, by an inspection of a sample of the ore, to determine when the bleaching operation is completed. After bleaching, the acid is discharged from the tank to be used again if strong enough, and the ore goes to the final washer where the last traces of clay and acid are removed.

ORE-DRYING AND MILLING.—For the purpose of drying the ore, the old style drying pan and the rotary dryer are both in use. After drying, the ore is ready for the mill, where it is reduced to the size of fine sand in rolls or vertical mills, and finally to an impalpable powder by means of buhr-stones. The ore is now ready for barreling and shipping.

The quantity of ore which can be bleached and milled in 24 hours varies greatly and depends on the content of iron in the ore and also its friability. Some of the harder ores require very careful milling in order that the last traces of grit may be removed. The average ore can be bleached in 24 hours, and twelve buhr-stones of 3½ ft. diameter will finish from 30 to 40 tons in the same amount of time.

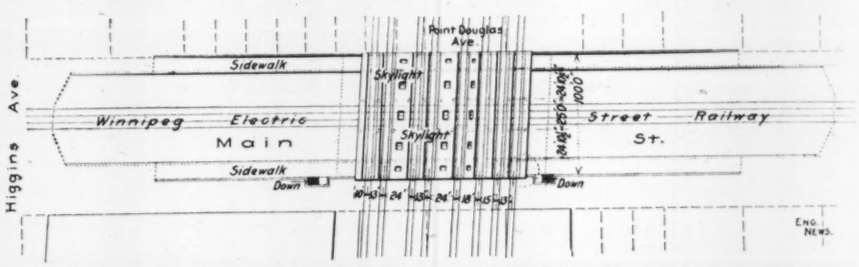

FIG. 2. PLAN OF MAIN ST. SUBWAY UNDER THE CANADIAN PACIFIC RY. TRACKS, AT WINNIPEG, MANITOBA.

A slight reddish or yellowish hue in the finished product, caused by incomplete bleachig or washing, is sometimes gotten rid of by an admixture of small amounts of blueing—from ½ oz. to 3 oz. to the ton. This practice is not general, however, as it is objectionable to some of the trade.

Slight variations from the practice outlined above are to be found. The most extreme of these involves the reduction of the ore to a powder before bleaching. This method obviously exposes more surface to the bleaching agent, but the subsequent operation of drying presents difficulties not met with in common practice.

FURNACE BLEACHING.—Manganese dioxide, a common impurity in barytes, cannot be removed by the use of sulphuric acid alone. For bleaching such ores a specially constructed furnace is used. The ore is ground so that it will pass through a 40-mesh screen, mixed with Chile nitre, salt and sulphuric acid, in proper proportions, and then transferred to the furnace. By the admixture of these chemicals aqua regia is formed and the oxides of iron and manganese are converted into chlorides.

These chlorides are very soluble in water and are entirely removed by washing and settling the ore in a series of tanks—generally three in num-

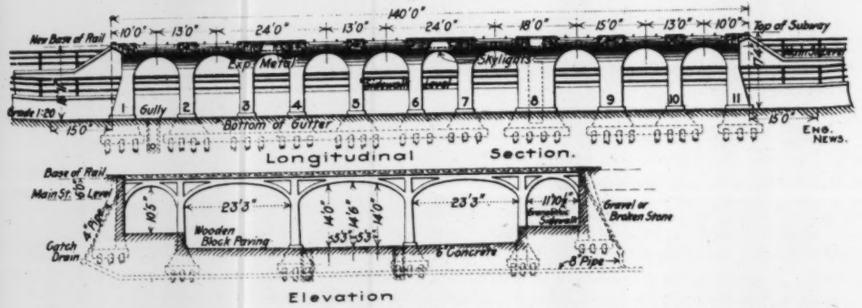

FIG. 3. ELEVATION AND SECTION OF CONCRETE SUBWAY AT MAIN ST., WINNIPEG.

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ber. The ore is then dried and put through buhr mills, the product being of a very high grade.

BLANC FIXE OR PERMANENT WHITE.—Blanc fixe is the name given to barium sulphate prepared in the following manner: The finely ground ore is mixed with about one-fourth its

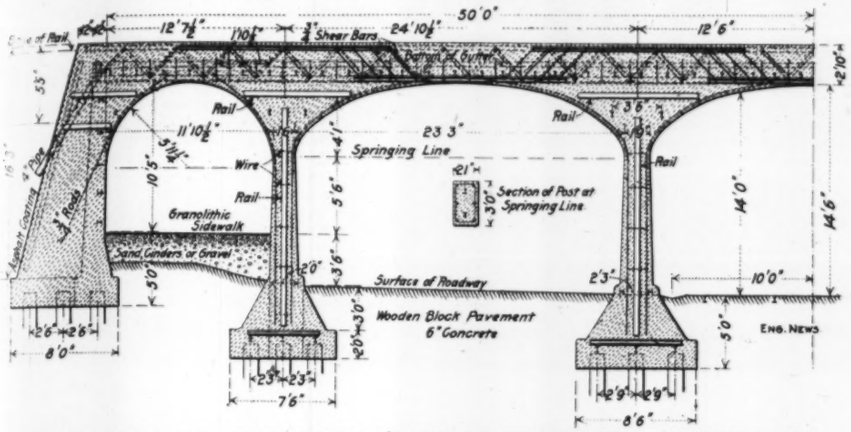


FIG. 4. HALF-TRANSVERSE SECTION OF MAIN ST. SUBWAY, SHOWING DETAILS OF REINFORCEMENT.

volume of coal, and subjected to a very high heat in a crucible or retort. The sulphate is reduced to the sulphide, which is dissolved in water and the solution filtered. From this solution barium sulphate is precipitated with sulphuric acid. The resulting product is absolutely white and entirely free from grit, and is more valuable than the best grade of barytes produced by any other process.

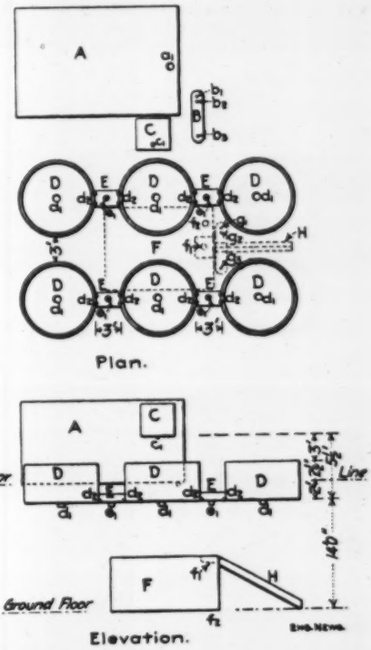
The following is a descriptive outline of some of the more important operations and apparatus employed in an up-to-date barytes mill, designed for the treatment of ore as it is usually mined: i. e., ores containing clay and stains of iron oxide as the only impurities.

It is imperative, in order to obtain the best results, that the ore be thoroughly dry before it is crushed; or else that it be wet crushed—that is, with a stream of water playing into the jaw of the crusher. Ore, as it is usually delivered to the mill may be damp or may be accompanied by considerable clay. During the winter months, especially, great difficulty will be experienced in getting such ore through the crusher. To avoid this difficulty wet crushing seems to be the simpler and cheaper alternative.

The ore is first (wet) crushed and then washed in a cut-flight washer. ⅝-in. pieces are screened off and the balance of the ore is sent to rolls to be reduced to this size. If the ore is washed before being sent to the rolls, there will be less loss in fines than would result should the washing be done after the rolls have completed their work.

The ore is now ready for the bleaching tanks, and is supplied to them by conveyors, so arranged that the discharge may be directed to any one of them. A circular bleaching tank is to be preferred for the reason that steaming can be more uniformly done than in a rectangular tank. A tank 4½ ft. high and 8 ft. in diameter is a convenient size to use. Calculating that the average crushed ore weighs 160 lbs. per cu. ft. a tank of this size charged to a height of 3½ ft. will contain approximately 14 tons. A larger size than this would require two steam inlets and this would complicate the arrangement of the lead pipes in the bottom of the tank. The tanks should be made of stout seasoned wood—cypress preferred—and should be well-braced on the outside, lined inside with heavy sheet lead, and provided with discharge orifices for acid and ore. In addition to the steam pipe, which is attached to the coil of lead pipe for the bottom of the tank, a water connection should also be made, so that either water or steam may be supplied from the perforations in the lead pipe. These perforations are best located at an angle of 45° off the vertical diameter of the pipe rather than directly on top, in order to prevent the clogging of the pipes with fines.

One of the greatest items of expense in the bleaching house is the steam used for heating and agitating the ore and acid during the operation



A Convenient Arrangement for Acid and Bleaching Tanks for the Treatment of Barytes.

A Acid storage tank.
a_1 Outlet for acid.
B Acid egg.
b_1 Inlet for acid.
b_2 Inlet for air.
b_3 Outlet for acid.
C Graduated acid tank.
c_1 Outlet for acid.
D Bleaching tanks.
d_1 Outlet for ore.
d_2 Outlet for sludge acid.
E Sludge acid pan.
e_1 Outlet for sludge acid leading to cludge tank.
F Sludge acid tank.
f_1 Orifice admitting acid to slude tank. When plug is inserted sludge acid flows off on trough H.
f_2 Outlet for sludge acid.
g_1 g_2 g_3 Sludge acid egg.
g_1 Inlet for sludge acid.
g_2 Inlet for air.
g_3 Outlet for sludge acid.
H Trough for carrying off sludge acid not strong enough to be used again.

of bleaching. This expense may be greatly reduced by the use of an ordinary injector, a mixture of air and steam being used instead of steam alone. The amount of air used may be easily regulated so that the acid will not be cooled. The temperature may be kept at 200° F, better agitation will result, the acid will suffer less dilution from condensing steam, and the steam consumption will be reduced by one-half.

Sulphuric acid (66° Beaume) may be obtained either in drums or in tank cars. An air compressor will make it possible to do away with the dangerous and costly method of drawing the acid from drums and pouring it into the tanks from buckets. By means of compressed air, a tank car can be unloaded in a short time into a large stor-

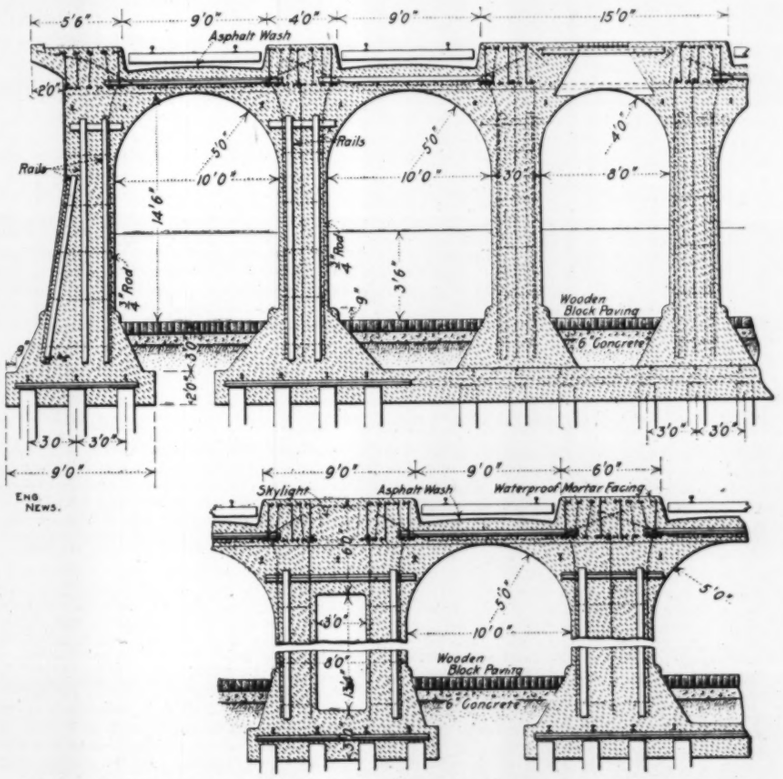


FIG. 5. LONGITUDINAL SECTIONS OF MAIN ST. SUBWAY, SHOWING DETAILS OF REINFORCEMENT.

age tank in the plant. From here it passes to an acid "egg" and by means of air is forced up into a graduated tank of about 4,000 lbs. capacity, located at some convenient point above the bleaching tanks. From here it can be readily supplied to the different tanks through a series of lead pipes.

The ore, after leaving the bleaching tank, goes through another cut flight washer (final washer) and then to a rotary dryer. The rotary dryer has a greater capacity than a drying pan using the same amount of fuel, and requires less than one-half the labor. From this point to the final reduction and barreling of the finished product the process is well understood and will be given no further attention here.

FINES.—In some friable ores there is a great loss in washing, due to "flouring" or pulverizing. This fine barytes will be carried off from the first washer, together with clay and other light particles of gangue. If the amount is found to be sufficiently great this "fines" may be treated very successfully on a shaking table, and sold as off-color product. Or it may be further treated in the furnace mentioned in this article, if thought advisable.

The fines from the final washer may be recovered in a series of settling boxes and disposed of with the heads from the shaking table.

SLUDGE ACID.—PIGMENT.—The acid, after having bleached one tank of ore, is often strong enough, with the addition of some fresh acid, to be used again. Such acid is referred to as "pickle" or "sludge". For the purpose of catching this sludge, a tank of suitable size may be constructed under the bleaching house. From this tank the sludge can be admitted to an acid egg and from there forced back to any of the bleaching tanks by means of compressed air.

If, in the first place, the ore runs high in iron and the sludge shows less than, say 5% of sulphuric acid, it may be run into a special tank containing scrap iron. When all of the acid is used up the solution may be treated with milk of lime in another tank. The resulting precipitate, when dried and roasted, makes a very good red pigment. The following are the reactions involved in this process:

$$Fe_2O_3 + 3H_2SO_4 = Fe_2(SO_4)_3 + 3H_2O.$$

$$3\,Ca(OH)_2 + Fe_2(SO_4)_3 = 2\,Fe(OH)_3 + 3\,CaSO_4$$

$$2\,Fe(OH)_3 + heat = Fe_2O_3 + 3\,H_2O.$$

Assume a sludge acid containing 50 grams of Fe per liter, which is equivalent to 178 grams of $Fe_2(SO_4)_3$, per liter, or 1.5 lbs. per gallon. Theoretically, 1,000 gallons would contain 1,500 lbs. of $Fe_2(SO_4)_3$, which would require for its precipitation 900 lbs. of 70 per cent lime. The yield would be 2,100 lbs. of pigment, containing 28.6 per cent. Fe_2O_3, assuming that the calcium sulphate remained unchanged. These figures have been very closely checked in practise.

OFFICE METHODS IN RAILWAY SURVEYS.*

By W. D. Gerber.†

It has been our practice to do as little work in the field with the location party as is possible. By this I mean that only working profiles and projecting plats are made in the field. As the survey progresses and a certain length of line, say 15 or 20 miles, has been definitely decided upon, the notes are returned to the office where they are carefully checked and from them the maps and profiles made.

In making up location maps, two sizes of drawings are made. One, which we term our location map, is on a scale of 2,000 ft. to the inch. On this map is shown the line, with all the notes relative to the alinement, together with the station numbers and land line ties; land lines, such as section and half-section lines or smaller sub-divisions where owned by different individuals, and the names of the property owners. No topography is shown on this map, except streams of considerable size or those which have water in them a considerable portion of the year.

The second map, which we call our right-of-way map, is made up in a continuous roll, 18 ins. wide, 500 ft. to the inch. On this are shown all notes relative to the location similar to the smaller map and also the right of way for which deeds are secured. Upon securing a deed for a piece of right-of-way, after the deed has been checked and recorded, a record is made on its proper parcel of land, giving the number of acres taken, price paid and the index of the filing. This then becomes a perfect office record of the right-of-way throughout the whole length of line plotted thereon. On this right-of-way map are also indicated the country highways, the location of any considerable improvement which lies close to the location, together with such topography as can be secured without resorting to taking notes for contour lines. It has not been our practice to show contour of lines through this territory. I doubt if any other road has gone to this extent in their map making.

Following the location and right-of-way maps, naturally comes the center line profile of the line. This profile is made on profile tracing paper with horizontal scale of 400 ft. to the inch and vertical scale of 20 ft. to the inch. The grade line is laid as it is expected to be built. The alinement is placed along the bottom, with the curve notes, so that one may readily see how much of a cut or fill is to be had or met with at any particular point, whether on tangent or curve. Our roadbed when finished is a 16-ft. crown for fills, and 20 ft. base with 4-ft. ditches on either side in cuts. This leaves a 12-ft. crown for road bed through cuts. Our specifications require the contractor to haul 300 ft. free; overhaul to be paid for at so much per yard for each 100 ft. of overhaul. In making up the estimate from the center line profile, these notes are naturally carried in mind.

PRELIMINARY ESTIMATE.—The method of working up the preliminary grading estimate is similar to that illustrated in the "Manual for Resident Engineers," by Molitor & Beard. Let us consider the section of profile herewith presented. The average end area, or rather the average center height, of any prismoid is found by scaled measurement, using a scale of 20 ft. to the inch. The cubical content is then found by reference to some table of level cuttings. To the excavation quantities thus found is then added all extra yardage on account of ditches, etc. With a vertical scale of 1-20-in. equal to 100 cu. yds. and with some horizontal line as a base line, construct the bulk or mass curves, starting at a point on the base line, directly under the grade point as at "O." Thus, by scaling and with reference to the table, we find that the embankment portion of the grade between Sta. 618 and Sta. 619 contains 85 cu. yds. The next 100-ft. prismoid contains 305 cu. yds., the next 2,800 cu. yds., the next 2,870 cu. yds., etc. Then beginning at "O" the bulk curve O, H, F, D, B is plotted for the embankment. Similarly the bulk curve O, G. E. C, A is plotted for the excavation portion of the work. Since the specifications require 300 ft. free haul, draw the horizontal line G H exactly 300 ft. long, measured with a scale of 400 ft. to the inch. The points G and H indicate the extreme limits of the free haul and the vertical distance between G H and O measured with scale of 1-20-in. to 100 cu. yds. represents the number of yards hauled free, which in this instance is 820. The points G and H are then projected into the profile and the free haul work is indicated as shown.

We will suppose it is desired to haul all of the excavation possible between Sta. 601 + 50 and Sta. 618 + 40 into the embankment; consequently we would proceed to find the free haul quantities of the left or east end of the cut in the manner just described, which gives us Sta. 599 + 80 and Sta. 602 + 80 as the respective limits of free haul with a quantity of 175 cu. yds. hauled free. This leaves us for excavation the quantity of material represented by the bulk curve D O C. Draw C D (a horizontal line through C) to an intersection of the embankment curve at D. This line then represents the length, when measured with proper scale, of extreme haul. By bisecting the vertical distance between the lines C D and G H and drawing through this bisecting point the horizontal line E F, to intersections of the mass curves at E and F, we obtain the center of bulk or mass. The length of this line represents the mean haul for the cubic yards of material represented by the mass curve between G and C, which is 11,060 cu. yds., or the total excavation yardage minus the sum of the two free haul quantities. The result of this work now stands as 11,060 cu. yds. of earth excavation hauled the average distance of 1,050 ft., but the specifications recite that each yard is to be hauled 300 ft. free, hence we have:

11,060 cu. yds.	hauled	1,055 ft.
11,060 "	" overhauled	(1,055 — 300) 755 ft.
83,503 "	" overhauled	100 ft.

We have now accounted for all the yardage in the excavation but are still short the amount of embankment represented by that portion of the embankment curve between D and B or 1,520 cu. yds. This amount of work must of necessity be borrow, and it is so indicated on the profile. Quantities which are wasted are also indicated in a similar manner.

It is very likely that one of the first questions which will arise regarding this method of estimating the grading on any piece of work will be "What is the percentage of error of such an estimate as compared to the actual quantities as computed from the cross-section notes"? We are just finishing some 15 miles of grading of which some seven miles has been completed. On comparison of the quantities found by this method of preliminary estimating and the quantities of actual work done, I find that the quantities of the latter overrun those of the former by 6.2%. On one particular mile, which is a fair representative of the general work found in this country, I find that the final quantities overrun the estimated quantities 2.5%. In conversation with several engineers of experience who have been familiar with this method of estimating, I have learned that they do not expect to find, under ordinary conditions, an error of more than 5 to 6%, which goes to show that this is a very fair and accurate method for preliminary estimation of quantities.

Following the preliminary estimate of grading comes, of course, the preparation of a preliminary estimate covering the total cost of the finished work. This is made up from the total estimated quantities of the different materials required, together with the current prices for such material and, as it resolves itself into the question of simple multiplication, requires no further discussion.

Upon the completion of each contract or any considerable portion thereof, a final profile is made and the hauled quantities computed in the same manner as described for the preliminary estimate, with the exception that the cubical content of each prismoid is found from the cross-section notes. This profile is a complete record of every yard of material handled and final payment for the grading is made from the information shown thereon.

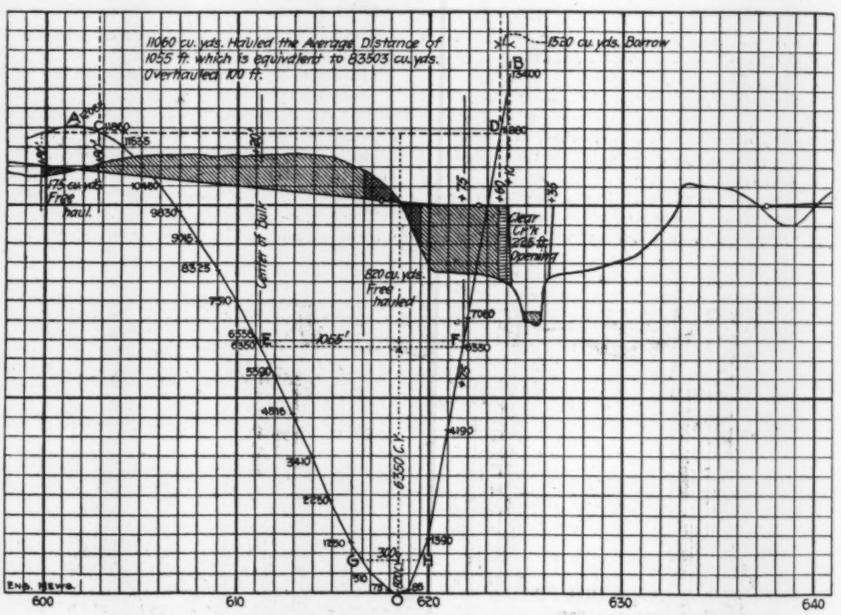


DIAGRAM FOR ESTIMATING OVERHAUL; DENVER, ENID & GULF RY.
W. D. Gerber, Chief Engineer.

*Abstract of a paper read at the annual meeting of the Illinois Society of Engineers and Surveyors, at Chicago, Jan. 20, 1905.

†Chief Engineer, Denver, Enid & Gulf Ry., Enid, Oklahoma.

THE WORLD'S LARGEST CONCRETE-STEEL ARCH BRIDGE: THE GRUENWALD BRIDGE AT MUNICH.

The largest concrete-steel bridge ever built was opened for traffic two months ago at Grünwald, near Munich, Germany. The structure contains two central arched spans, which exceed the span of the well-famous Cabin John stone arch. The mark set by the latter bridge has only recently been exceeded in stone-arch construction, and now the new building material, concrete-steel, has approached closely the maximum lengths of stone arches. In the following description of the Grünwald bridge the small thickness of the arch ring, and the lightness of the whole construction, should be particularly noted, as they define results which have scarcely been reached with stone.

The Grünwald bridge comprises two main spans, which are three-hinged arches, and a number of small girder spans, all of reinforced concrete. The roadway of the main spans and its supporting posts are of the same material. Thus the structure contains several distinct points of interest. But it should be expressly remarked that while the girder spans, the roadway and the roadway posts are true reinforced concrete, depending vitally upon the contained steel, on the other hand the main arches were designed on the basis of the strength of their concrete, without any calculated stressing in the steel. The steel was added merely as a security against tensile stresses that *might* arise by accident, though the arch was so planned that every conceivable loading is safely provided for in the concrete.

GENERAL DESCRIPTION AND DIMENSIONS.—The Grünwald bridge has two arched river spans, each 70 m. (230 ft.) in span and 12.8 m (42 ft.) in rise, center to center of hinges, and five girder spans each 8.5 m (28 ft.) in the clear. Of the girder spans, four are on the left bank and one is on the right bank of the river. The roadway is 9.2 m (30 ft.) wide over all, containing a roadway 5 m. wide and two sidewalks each 1.5 m. wide in the clear. The arches themselves are 8 m. (26.2 ft.) wide. The total length of the bridge is about 220 m. (720 ft.)

The structure is a highway bridge, and was designed for a live load of 82 lbs. per sq. ft. uniform, and a 22-ton steam road roller. It will be appreciated that in spite of the extreme lightness of the structure, the live load is small compared with the dead load.

The main arches were made three-hinged for a combination of reasons, chief among which are (1) that a very short time was available for the calculation and design, too little for the proper treatment of a statically indeterminate structure; and (2) that the character of the subsoil was known only in a general way, so that it was important to use a design which would minimize the effects of possible yielding of the abutments.

The "open spandrel" style of construction was adopted to reduce the dead load as much as possible. In large arches the dead load is so great a part of the total load that it virtually determines the required section of arch ring. Moreover, for the same reason the stiffening effect of solid spandrels is of little importance in large arches. On both grounds the "open spandrel" construction was called for, and in the structure as built this style is carried to its extreme possibilities by subdividing the usual transverse walls into independent posts, on which rests the roadway platform. There are five lines of these posts in the width of 8 m., that is, they are spaced 2 m. apart transversely; longitudinally the posts are 4 m. apart.

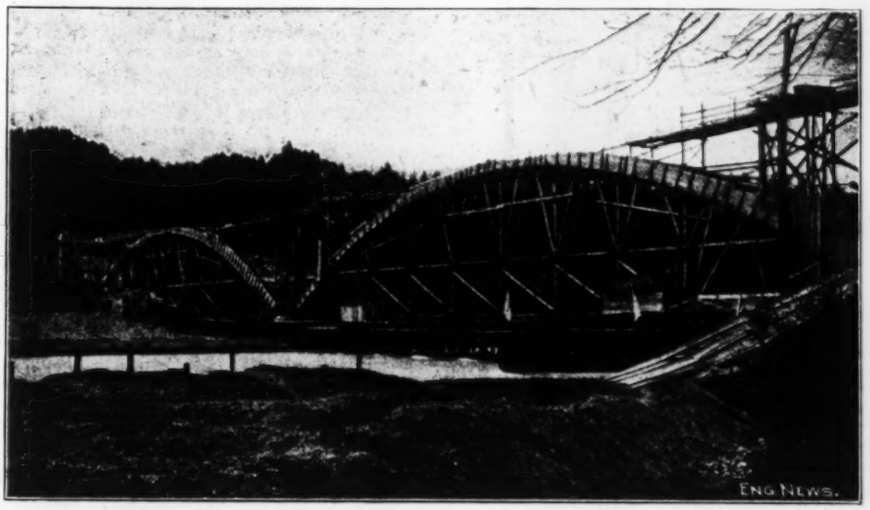


FIG. 1. VIEW OF CONCRETE-STEEL ARCH BRIDGE OVER THE ISAR, AT GRUENWALD, NEAR MUNICH, BAVARIA.

The accompanying Fig. 2 shows outline elevation, plan and sections of the Grünwald bridge, from which the principal facts about the structure as designed may be read without further explanation. The outline of the arch is not a mathematical curve, but was determined, by computation, such as to give the least variation of moment in the different cross-sections. The arch ring is remarkably thin. At the crown it is 30 ins., at the springing lines 36 ins., and at the thickest portion, at about the quarter points, it is but 48 ins. deep.

The roadway rises on a 1% grade from each main abutment toward the midstream pier, while landward of the main abutments the grade is only 0.5%.

The disposition of the girder shore spans is also apparent from Fig. 1. Their detail construction, as also that of the arch, will be noted farther on.

On the basis of an assumed size and shape of arch rib, the dead load was approximately computed and the dead-load pressure line was drawn. The dead-load and live-load moments were then computed for the various cross-sections. Now, if the eccentricity of the dead-load thrust be called e, and the depth of rib be called h, we can write the dead-load fiber stress equations for normal thrust P as follows:

$$S_1 \frac{bh^2}{6} = P\left(\frac{h}{6} - e\right)$$

$$S_2 \frac{bh^2}{6} = P\left(\frac{h}{6} + e\right)$$

which equations are readily verified.

Then, considering the live-load moment as being unaffected by small changes in the axis of the arch, and neglecting the live-load thrust, we can write

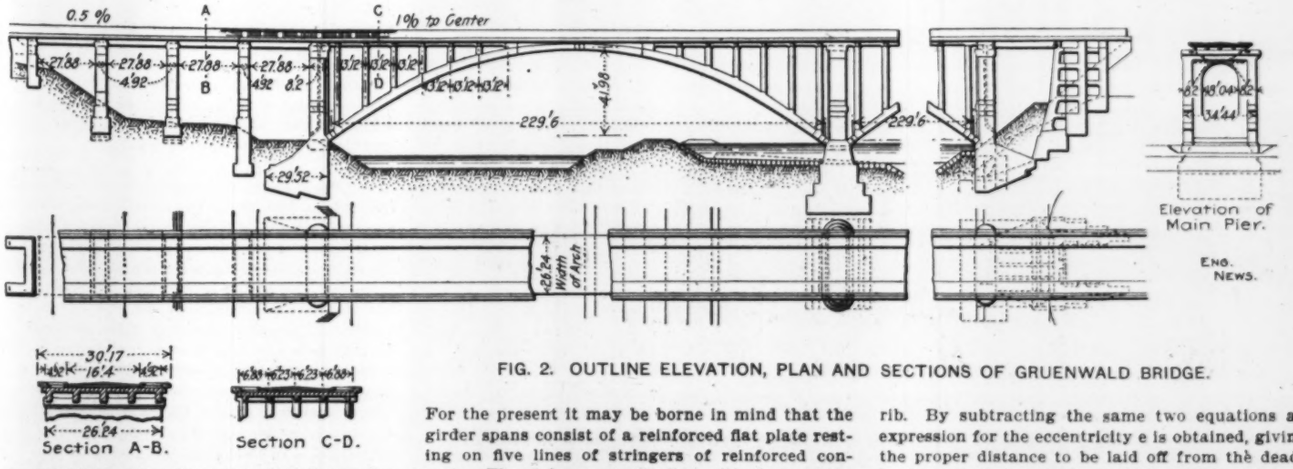
$$\text{Max. } S_1 \frac{bh^2}{6} = P\left(\frac{h}{6} - e\right) + \text{negative L. L Mom.}$$

and

$$\text{Max. } S_2 \frac{bh^2}{6} = P\left(\frac{h}{6} + e\right) + \text{positive L. L. Mom.}$$

By adding these equations, and making Max. S_1 = Max. S_2 = allowed unit stress, an equation in h is obtained which gives the required thickness of rib. By subtracting the same two equations an expression for the eccentricity e is obtained, giving the proper distance to be laid off from the dead-load pressure line to locate the axis of the arch rib. The result may be used as a basis for recalculation, to give a second approximation. It will be noticed that this interesting method of design involves the assumption of a linear stress strain curve for the concrete.

This method of calculation was used to give the form of the arch rib and its thickness. The stresses were then recomputed for obtaining the minimum stresses, the maximum stresses being, of



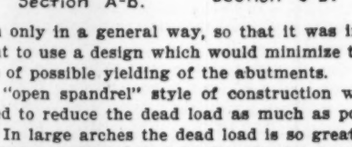


FIG. 2. OUTLINE ELEVATION, PLAN AND SECTIONS OF GRUENWALD BRIDGE.

For the present it may be borne in mind that the girder spans consist of a reinforced flat plate resting on five lines of stringers of reinforced concrete. The stringers are in line with those of the roadway over the arches, where the stringers correspond with the five lines of posts.

DESIGN AND CALCULATION.—As already noted, the assumed loading was a uniformly distributed loading of 400 kg. per sq. m. (82 lbs. per sq. ft.), and a concentrated load consisting of a 22-ton road roller. The permissible compression in the concrete was taken at 36 kg. per sq. cm. (510 lbs. per sq. in.).

course, 36 kg. per sq. cm. as assumed in the analysis. The minimum stresses were found to range from 20 down to 2.1 kg. per sq. cm. (285 to 30 lbs. per sq. in.). Now it may be calculated that if the actual center line of the arch should deviate by about 2 ins. from its computed position, the least minimum stress would be just balanced, and any greater deviation would give opportunity for

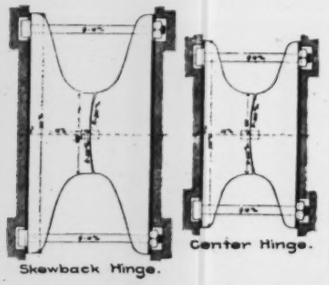


Fig. 3. Steel Hinges of Gruenwald Bridge.

incipient tension at some point of the arch. To take care of such possible tension, the designer of the bridge states, a steel reinforcement was provided.

The preceding refers wholly to the design of the arch ribs. The roadway plate, stringers and posts, which are "straight" concrete-steel work, were designed as such, the steel being computed for the required tension in the case of the beamwork and put in more or less arbitrarily in the case of the posts, as usual.

FOUNDATIONS AND PIERS.—The main foundations for the bridge were carried down by excavation in open wells to a hard marl-like substratum, which could safely support a loading of about 5 tons per sq. ft. The piers were carried up from this level in solid concrete, apparently without reinforcement. The main piers, which are 2.5 m. (8.2 ft.) thick, as well as the piers of the girder spans, 1.5 m. (4.9 ft.) thick, have a central opening in the upper part, as a means of saving weight and material. The top of these openings is arched, and reinforcing rods are used in the concrete over the arches to provide for the stringer reactions. The section shown in Fig. 2 indicates the location of these rods.

MAIN ARCHES.—As already noted, the main arches are three-hinged. The hinges are steel castings, with a convex-concave rolling surface. The construction of the hinges in detail is shown in Fig. 3. The convex face has a radius of 8 ins., the concave face a radius of 10 ins. A dowel at the center provides security against such an accident as befell the Maximilian stone-arch bridge, which is located only a short distance away. It is to be noted, moreover, that the radii of the joint faces are very much smaller than in the Maximilian bridge (the radii in that bridge are 1 ft. for the convex face and 10 ft. for the concave face; see a description of the bridge and its failure in Eng. News of Oct. 27, 1904). This fact alone gives much greater security against slipping of the hinges.

The designer of the bridge remarks, concerning the selection of steel casting as material for the hinges, that lead-plate hinges, because of their imperfect hinge action, were not considered, while cut-stone hinges were held to be undesirable on account of the much larger radii of contact faces which they require, as well as because of their greater rolling friction. Large radii are objectionable because any rolling of the joints produces a large shifting of the line of contact, so even with careful work it may happen that after striking centers the line of contact is very materially out of center of the hinge faces; such a condition would, of course, produce stresses in the end sections very different from those computed in the design.

The pressure allowed on the lead plate back of the hinge casting was 1,400 lbs. per sq. in. As this is greater than the allowed compression in the arch concrete, it was necessary to use blocks of special strength back of the hinges. Granite blocks were originally planned to be used here, but for reasons of economy it was later decided to use carefully made blocks of reinforced concrete. The blocks were molded in planed cast-iron molds, and were provided with transverse reinforcing bars to prevent splitting; these bars extend at right angles to the general direction of the hinge joints, and are distributed uniformly through the thickness of the block. Each block is 79 cm. long in the direction of the hinge joint, so that, with 1 cm. joints, there is one block under each hinge casting.

The arch ring itself requires no further explanation, except as to its reinforcement. There are eighteen longitudinal 28-mm. (1.1-in.) steel rods, nine each at the top and bottom of the ring, tied together at intervals of about 3 ft. by ¼-in. stirrups. Round steel was used throughout the structure.

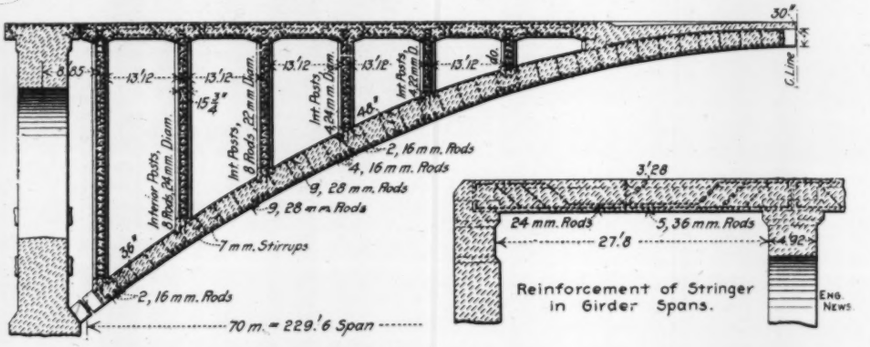
In addition to this main reinforcement, there is special reinforcement under each transverse line of posts, in order to distribute the concentrated column loads more effectually over the 8-m. width of the arch ring. Under each line of posts are two transverse rods 16 mm. (⅝-in.) in diameter near the top of the arch rib, and four rods of the same size near the bottom of the rib. This special reinforcement, as well as the main reinforcement, is shown clearly in the reinforcement diagram, Fig. 4. It will be noted that only four distributing rods are shown under the last row of posts toward the skewbacks; this is because these posts, being longest, are so connected by cross struts for stability as to form virtually a transverse wall, requiring less distributing action in the arch rib itself.

POSTS, ROADWAY AND APPROACHES.—The posts under the roadway support each an area of 4 m. by 2 m., as has been mentioned. Their length varies from about 5 ft. to about 38 ft. All interior posts (i. e., excepting those in the side faces of the structure) were made of equal size, 40 × 40 cm. (16 × 16 ins.) in section. Their reinforcement consists of longitudinal rods tied together by 7-mm. (.275-in.) stirrups at intervals of 35 cm. (14 ins.). The longitudinal reinforcement varies from 8 rods 24 mm. (15-16-in.) in diameter, to 4 rods 22 mm. (⅞-in.) in diameter, as is noted in Fig. 4. The posts in the outside rows were widened out at the outer face to a T-section, so as to present a wider face (70 cm. = 28 ins.) in side elevation. Their reinforcement varies from 8 rods 24 mm. (13-16-in.) in diameter to 8 rods 18 mm. (11-16-in.) in diameter.

The reinforcement of the posts is extended some 40 to 50 cm. (16 to 20 ins.) downward into the concrete of the arch ring.

The roadway over the arch consists of a flat plate 2 m. (6.6 ft.) in span c. to c., resting on five lines of longitudinal stringers spanning 4 m. (13.1 ft.) c. to c. The roadway plate is the same in span and in detail construction over arches and approach spans, but the stringers in the approaches span 8.5 m. (27.9 ft.) in the clear, and hence are larger than those over the main arches.

The dimensions and reinforcement of the floor-plate and stringers are shown by Fig. 5, in connection with Fig. 4. The floor-plate is 8 ins. thick, and is reinforced in each 4 m. panel by twelve top rods, eight of 10 mm. (⅜-in.) diameter and four of 12 mm. (½-in.) diameter, and ten bottom rods, six of 12 mm. (½-in.) and four of 14 mm. (9-16-in.) diameter. Six of the bottom rods are bent diagonally upwards near the quarter points and continue in the plane of the upper reinforcement, so that over the stringers the plate has 18 top rods, averaging 7-16-in. diameter, in the 4-m. panel width. This is to provide for the negative moment over the supports, the plate having been calculated as a continuous beam.



Reinforcement of Arch-Ring and Posts.

FIG. 4. REINFORCEMENT DIAGRAM OF 230-FT. ARCH.

The floor stringers over the main arches are 25 cm. wide by 40 cm. deep (10 × 16 ins.) below the floor-plate, and have their reinforcement arranged similarly to that of the plate. Each stringer has in the main four bottom rods, averaging 22 mm. (⅞-in.), and two top rods of 20 mm. (13-16-in.) diameter. Over the supporting posts half of the bottom rods are bent upward and lie in the top surface, increasing the top reinforcement. The bottom rods of adjacent 4-m. spans lap past each other, so that the reduced bottom reinforcement is again made up. Since in a continuous beam the moments over the supports are greater than the center moments, the stringers were slightly bracketed downward at the posts, so as to give greater effective depth at the points of maximum moment.

The stringers of the approaches are 40 cm. by 80 cm. (16 × 32 ins.) below the floor-plate. They are reinforced with six bottom rods, one of which is 24 mm. (15-16 in.), while the other five are 36 mm. (1⅜-in.) in diameter. Three of these rods are straight, while the other three are bent up at the quarters, as shown in Fig. 5. These girders were not calculated or built continuous over the

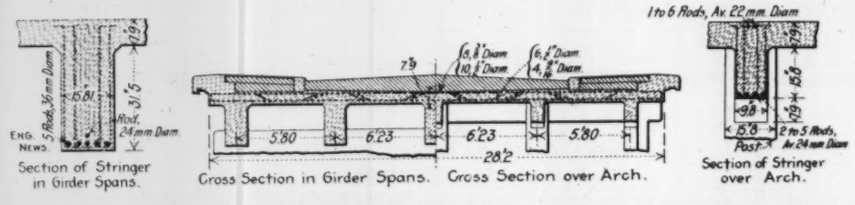


FIG. 5. REINFORCED CONCRETE ROADWAY STRINGERS, FLOOR-PLATE, AND APPROACH GIRDERS.

The hinge castings were made in lengths of 78 cm. (31½ ins.), and adjoining pairs were spaced 2 cm. apart in the clear. This gives ten pairs of hinges in each joint. The back face of each casting was planed, and a lead plate about ⅛-in. thick was placed between the casting and the arch block against which it abuts. Four ⅜-in. bolts served to hold together the two castings of each pair during erection.

piers, so that extra top reinforcement was not required.

In the reinforcement of this structure, the ends of the rods were generally bent at right angles to hook into the concrete. Stirrups were used in the stringers and girders as well as in the posts.

CENTERING AND WORK OF CONSTRUCTION.—The general plan of building the large arch involved three features: (1) Making the centering as rigid as possible, so as to minimize the deflection; (2) Putting in the arch concrete in such an order that the tendency of the actual deflections to produce cracks or internal stresses would be minimized; (3) Constructing the complete bridge, i. e., arches, posts, stringers, floor-plate and paving, before striking centers, so that the bridge would be in its final dead-load condition as soon as the centers were removed.

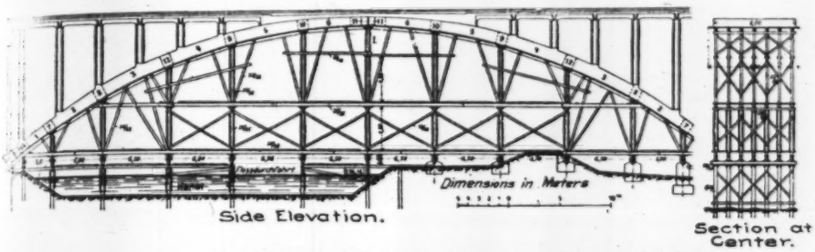


FIG. 6. CENTERING FOR 230-FT. ARCH, GRUENWALD BRIDGE.

The arches were built on timber center-frames supported on piling at all panel points, 6.5 m. (21.3 ft.) apart. The construction of the centers is shown in Fig. 6. A source of deflections very frequently found in arch centers, namely, the cross-fiber compression of sills and caps at the post bearings, was taken care of by using bearing-pieces made of steel channels between post and sill or cap. These channels were made long enough to distribute the post loading over a sufficient area of the horizontal timber to keep the transverse crushing stress far within safe limits. Seven center-frames were used in each span, spaced about 4 ft. apart.

The centers were supported by sand-boxes at each pile, except that oak wedges were used at the two panel-points nearest each skewback. Thus, seven posts in each center-frame had sand-boxes. Fig. 7 shows the construction of one of the sand-boxes. The box itself is made of wrought-iron plate, and the plunger is of oak.

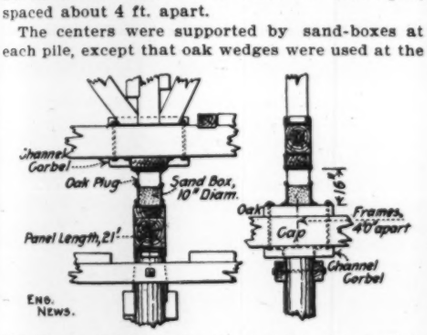


Fig. 7. Sand-Box Under Panel-Point of Centering.

The hinges and the reinforced concrete hinge-blocks were placed on the centering first. Both rows of hinge-blocks in each joint rested on the centers, which arrangement avoided any possible slip of one-half of the hinge relative to the other during erection.

The concreting of the arch ring was carried on in separate transverse courses, whose distribution was so planned as to avoid the formation of long, continuous sections up to as late a time as possible. In Fig. 6 the order of concreting is indicated by numbers placed on the arch sections. The concrete in each full panel was placed before the joining blocks over the posts were put in, and the order of the latter is such as to make the three last closures at crown, quarter-points, and skewbacks in succession.

The concrete mixture used was: 1 part Blaubeurer Portland, 2 parts river sand, 4 parts river gravel.

After the superstructure was complete, when the arch-concrete was about three months old, centers were slacked in the following procedure: About 15 cu. ins. sand was drawn out of each sand-box of the middle row of each span. After plugging the openings again, these boxes were tapped with a hammer, which caused the plungers to settle down. Then the adjoining row on either side of the center, in both spans, was similarly treated; then the next row, and then the next or final row of sand-boxes. Returning now to the middle, the same procedure was gone through again, and thus repeatedly until the levels on the bridge showed that the elastic compression was going out of the centers, i. e., that the load was nearly all on the arches themselves. Now the wedges under the last two panel points of the centering toward the abutments were slacked off, and the centers were then rapidly lowered down and removed.

As evidence of the stiffness of the centers and of the bridge, the following figures are important: The centering originally was built with a camber of 10 cm. (4 ins.). With the full dead load on, the deflection was 1.7 cm.; from this time until slacking centers there was no further deflection. The drop of the crown of the arch, while slacking centers, was 0.65 and 1.0 cm. respectively for the two spans. Thus the total drop, from the unloaded position of the centers, was only about 1 in., and the arches now are 3 ins. too high at the crown.

WATERPROOFING, EXPANSION, AND FINISH.—The back or top face of the two large arches was covered with a smooth coating of rich cement, as waterproofing. The floor-plate of the bridge was waterproofed with felt and asphalt.

The roadway is built with a slip-joint over each hinge-joint of the arches. These expansion joints are covered with steel plate in the usual way.

The surface of the concrete, in all parts of the structure, was left as it came from the forms, without any attempt at finishing except that fins produced at the points of the mold planks were chiseled off. The railing was molded in place, and was left without any finishing work.

After completion of the structure, the hinge-joints of the arches were filled solid with cement mortar, except that a sheet of asphalted felt was placed in the plane of contact, to maintain the hinge-joint. This is intended to protect the steel hinges from the weather and yet preserve the mobility of the joints.

The construction of the bridge was begun in the fall of 1903. Centers were slacked early in August, 1904. A load-test was made on Nov. 15, and the bridge was opened for traffic on Nov. 20, 1904.

The bridge was built by the Eisenbetongesellschaft München, for the city of Munich; it was designed by Professor Mörsch, of Munich. The total cost of the structure was about 260,000 M., or $65,000. The information and illustrations used in preparing the above article are from an article by Professor Mörsch in the "Schweizerische Bauzeitung."

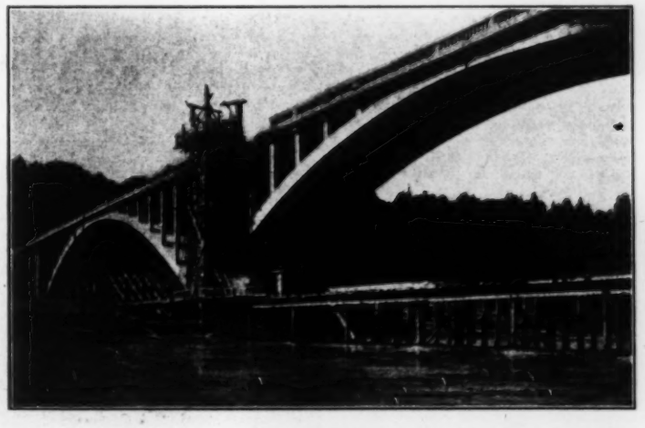


FIG. 8. GRUENWALD BRIDGE AFTER REMOVAL OF CENTERING IN BOTH SPANS.

THE COST OF PAVEMENT OPENINGS IN NEW YORK CITY.*

By Wisner Martin,† M. Am. Soc. C. E.

The objections to pavement openings may be classed as follows: first, they are obstructions to traffic; second, dangerous to life and limb; third, detrimental to health; fourth, unsightly and uncleanly; fifth, costly to make and to repair. The cost of the first four objections cannot be estimated in dollars; it is, however, great. Referring to the obstruction to traffic alone, Mr. Hayward, the eminent engineer of the London County Council, I believe, has given it as his opinion that it would pay to pave the Strand in London, with gold, if, by so doing, it would prevent any opening of the pavement. Perhaps it would be worth as much to get rid of each of the objections classed second, third and fourth. In other words, I take it that Mr. Hayward means that the advantages to be gained by doing away with all pavement openings by, for instance, the construction of pipe galleries, are so great that they cannot be estimated in dollars, and I think most of us will agree with him. Referring to the fifth objection, which is the only one which can be figured in dollars and which is practically the subject assigned to me, I have prepared the following figures from the carefully kept records of this office.

During the year 1901, pavement openings were made in Manhattan, in the different kinds of pavement in all of the streets as given below:‡ for house services and repairs, for electricity, gas, steam and water, 45,000 sq. yds. of pavement were taken up, of which 28,000 sq. yds. were asphalt and granite on concrete foundation. The cost of removing the pavement and making the excavations and of refilling the cuts and restoring the pavement was $210,000. Of this amount, $128,000 was for cuts in asphalt and $49,000 for cuts in granite on concrete. The number of openings was a little over 20,000.

To lay new mains and overhaul old ones, work which requires the opening of continuous trenches, each one of which was more than 80 ft. long, 100 miles in length of pavement was taken up at a cost for cutting the pavement, making the excavation, refilling the excavation and restoring the pavement, of $346,000. These figures are conservative. The year selected, as already stated, was one in which the number of openings was less than they have been since, and the estimates of cost are low. For instance, in 1904, for services and repairs there were 29,000 openings.

Efforts are always made by the engineers of the companies and by the city engineers to make improvements in design of sub-surface structures and in other ways, in order to prevent the opening of the pavements as much as possible. In Madison Avenue from 60th to 66th Street, in one year, the N. Y. Steam Co. made 52 separate openings for repairs, which is an average of one every week. This led to a radical reform in the method of laying the steam heating mains. Many improvements were carried out and the number of cuts necessary to make repairs has been constantly reduced year by year. Another case illustrating this point is shown by the improvements made by the Telephone Co. Twenty-five years ago a 3-in. electric cable duct could carry only 40 circuits or 40 separate telephone lines. Now they carry 400 and, in special instances, 500 and even 600. One can imagine how much tearing up of the pavements would have had to be done in order to increase the size of the

*A paper read before the Municipal Club of Brooklyn, Jan. 24, 1905.

†Principal Assistant Engineer, Bureau of Highways for Manhattan, Park Row Bldg., New York.

‡During that year the number of openings for the purpose of making house service connections and repairs was unusually small, and I select that year because I have the figures conveniently at hand.

main conduits to supply the enormously increased demand for telephone lines in lower Manhattan if this improvement had not been made. Even with this great improvement, however, last year the Telephone Company had to lay 96 additional ducts the whole length of Church Street to obtain sufficient room for the trunk lines between exchanges.

We still have many shortcomings, however. No method has been discovered of locating a gas leak or a water

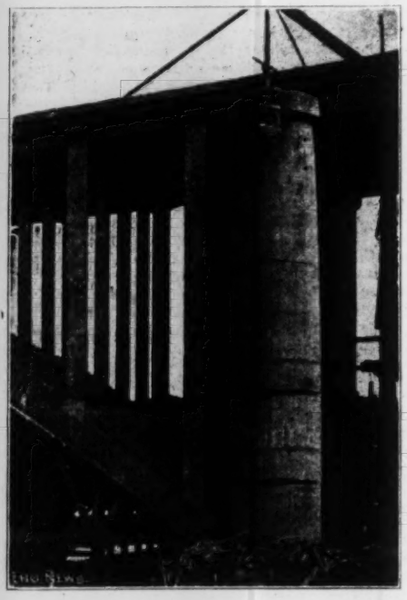


Fig. 9. View of Shore Pier of Gruenwald Bridge.

leak except by cutting a hole at each joint of the main. Sometimes a block or two has to be dug up in this manner at points every 12 ft. apart. These leaks, too, occur more frequently in the winter time when it is almost impossible to have the holes properly refilled and to relay the pavement, especially asphalt.

This difficulty of restoring the pavement over cuts made in the winter time has compelled the adoption of the rule, not to issue permits from the first of December to the first of March except in cases where it is absolutely necessary. This means that if a storekeeper wishes to change his system of lighting from gas to electricity, he must do it at another time of the year than between the dates mentioned. If there is any possibility of a customer of the telephone, electric light, gas or other company getting along under the old conditions during this winter season, he is compelled to do so. Every application for a new service to a house asked for at this time is referred to an inspector and a careful investigation is made as to the necessity. The effect of this rule is shown by our weekly reports of the number of permits issued; for instance, during the week ending July 14, 1904, 375 permits were issued, 25 of which were for mains, 86 for services, and 259 for repairs. For the week ending Jan. 12, 1905, 138 permits were issued, one for a new main, 11 for new services and 123 for repairs. This new main was issued to provide electric power for a large, new factory employing 600 hands, which was opened for business on Jan. 16.

I think it will be seen by what I have stated that considerable precaution and thoughtfulness are given to the matter of preventing, as far as possible, the tearing up of the pavements. Unfortunately, however, the number of openings does not decrease materially from year to year; in fact, with the extension of the asphalt pavements, they seem to increase. Although before an asphalt pavement is laid in any block all of the companies are required to put their substructures in good repair, after a little while enlarged mains are required, many new house connections, leaks occur which require numerous openings, and, by actual count, the whole number of cuts increases. The deterioration of the companies' plant is also a large factor.

Another subject which presents many difficulties is that of the mapping of all of the lines of pipes and electric conduits, which are continually being laid under the pavements. In most of the avenues and streets in Manhattan, before a new line can be laid, it is necessary to dig test pits to find out what is already in the streets. These are made before the new work is laid out and have to be repaired and then the pavements are opened again for the trenches for the new work, thus multiplying the cuts. Sometimes it is necessary to dig test pits on both sides of a street before room is found for a new line or for a larger sized water main, many of which are being required because of the erection of the many tall buildings. On Fifth Avenue it was found that there were six gas mains on each side. These had been laid by each of the individual companies which had franchises and which afterward were consolidated into one company.

I believe that this crowded condition of some of the roadways of our streets from curb to curb will force the solution of the problem, which is becoming more urgent every day, by the use of pipe galleries. They have been tried in London as early as 1860, in Nottingham, in St. Helens, in Milan, and appear to be wholly satisfactory. The Borough Engineer of Nottingham stated in a paper read before the Association of the Municipal and County Engineers that the money to construct a subway or pipe gallery there was borrowed at 5%, and that the rentals of space to the companies covered about 3½%, leaving a net annual cost to the city of the difference. He maintains, however, that this amount was more than covered by the saving in pavement rentals and repairs, length of life of pipes and eliminated leakage. I believe that the financial results would be much better in Manhattan. Money can be borrowed at 3½%, and the rental alone should equal this.

He also states that "in Victoria Street, in which is situated the general post office, there are, besides the gas and water pipes and connections, no less than six pipes containing telegraph wires and that not one single stone was disturbed in the carriageway for 25 years and, in that time, not one single penny was spent on repairs in this street,"—a stone paved street, of course. There appears to be no danger from putting gas mains and high and low tension electric wires in the same gallery. A gallery containing these can be ventilated and lighted by electricity and constantly patrolled by men in search of leaks which can be immediately repaired when found; the mains are kept in perfect condition so that their life is greatly prolonged; connections can be made to the houses without pavement cuts; and increased supplies easily provided for.

An advantage of pipe galleries which is sometimes overlooked but which is very important is that street boxes can be done away with. These cast-iron frames and covers are solid and rigid in the roadway and the wear is very slight on them compared to that on asphalt, for instance, surrounding them. The asphalt is very hard to maintain adjacent to these boxes. The boxes also are a nuisance as they are noisy. The wheels of a wagon with steel tires run quietly along the asphalt until they strike a box when a loud noise is made which is very troublesome, especially in a residential neighborhood.

A system of galleries was developed by Mr. St. John Clarke, one of the Rapid Transit Commission's engineering staff in consultation with the writer, to be tried somewhat as an experiment on Broadway from Ann Street to Morris Street. It was to be built in connection with the extension of the Rapid Transit Subway. It was designed so that all of the advantages of the European pipe galleries were obtained. There were no openings through street boxes set in the roadway pavement, all the necessary openings being provided for on the sidewalk. Money was appropriated to carry out the project by the Board of Aldermen and it seemed as though at last a start would be made in the right direction, but the matter was brought to a standstill by a taxpayer's suit. I am told that the plaintiff was successful in obtaining an injunction, not because he could show that the improvement was wasteful of public money, was insufficient or otherwise, but that it was about to be carried out under the authority of the wrong city official, the Charter providing that such work should be under the cognizance and control of the Commissioner of Water Supply, Gas and Electricity, and not the Borough President. It is deplorable that pipe gallaries could not have been provided along the main line of the rapid transit subway, at least below 42d St. when the subway was constructed. It is still more to be regretted that this short line has been obstructed.

As an illustration of some discouraging conditions now existing, this case occurred on New Elm St.: After the rapid transit subway was completed, the old pipes and electric mains were relaid on top of the steel structure of the subway and the earth filled in. The Borough President took advantage of this opportunity to lay a new granite pavement on a concrete foundation, but before the contractor had completed his job the water department asked for permits to dig test pits along this whole line in order to locate these same pipes and electric conduits. It seems it had made no record of the position in which they were placed only a few months before. The purpose of the test pits was to determine the location of the main pipes of the new salt water fire system.

There seems to be some opposition to the construction of pipe galleries in unexpected quarters so that some education will have to be accomplished before they will be a fact. Perhaps the laying of the distributing mains for the new city lighting plant will help in the construction of pipe galleries. However, that may be, it seems now as if they were the only solution of this tremendous problem and that they must inevitably come.

THE LARGEST PIER on the Atlantic seaboard was completed last year by the Baltimore & Ohio R. R. Co. at its Baltimore water front terminal. The pier is 800 ft. long and 127 ft. in width.

A LOCOMOTIVE BOILER EXPLOSION.

As briefly noted in our issue of Feb. 9, a serious locomotive boiler explosion occurred near Utica, N. Y., wrecking the passenger train which the locomotive was drawing and also an express train which was passing on the opposite track at the moment of explosion.

The photographs herewith reproduced were taken by Mr. Roy R. Brackett, a student in the senior class in Mechanical Engineering in the College of Applied Science of Syracuse University. They show the exploded boiler after it had been loaded on a flat car to be hauled away. The barrel of the boiler is seen to be intact, but the stays were pulled out of the internal firebox and it was forced downward, as shown in the views. The entire boiler was torn from the running gear of the locomotive and thrown into the air, and, by its own momentum as part of the moving train, it was carried forward a distance of several hundred feet before it struck the ground. It finally landed on the opposite track from that on which its locomotive was traveling, and by great good fortune behind the rear car of the train on that track. The doubling up of the front end of the boiler, seen in these photographs, was probably due to that striking the ground first.

The explosion was caused by low water, the overheating of the crown sheet permitting the crown stays to pull through the sheet. Strictly speaking, it may be said that the boiler did not explode; but the results of the crown-sheet coming down were so serious and might have been so much more serious had the flying boiler fallen on one of the cars, that the accident deserves attention, in view of the prevalent opinion among master mechanics that the failure of a boiler by an overheated crown sheet is not a matter involving extreme danger.

The engineer and fireman were killed by the explosion, but no other person on either train was killed or fatally injured.

VIEWS OF EXPLODED BOILER OF NEW YORK CENTRAL & HUDSON RIVER R. R., FEB. 4, 1905.

ENGINEERING NEWS

A Journal of Civil, Mechanical, Mining and Electrical Engineering.

Entered at the New York Post-Office as Second-Class Matter. Published every Thursday at St. Paul Building, 220 Broadway, New York, by

THE ENGINEERING NEWS PUBLISHING CO.

PUBLICATION OFFICE, 220 BROADWAY, NEW YORK.
CHICAGO OFFICE, 1636 MONADNOCK BLOCK.
BOSTON OFFICE, 170 SUMMER ST.
ATLANTA OFFICE, AUSTELL BUILDING.

FOREIGN AGENTS.
RUDOLF MOSSE, BERLIN and HAMBURG, GERMANY.
JAMES F. JONES, & CIE, 31 Rue de Faubourg Montmartre, PARIS.

SUBSCRIPTION RATES: United States, Canada and Mexico: One Year, $5.00; 6 months, $2.50; 10 weeks, $1.00. To all other countries in the Postal Union: Regular Edition, One Year, $9.00; Thin Paper Edition, One Year, $5.00 (21 shillings). SINGLE COPIES, 15c.

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The increasing consumption of rubber and the consequent rapid advance in price is a matter worth the attention of engineers engaged in mechanical design, where the use of parts made of rubber is occasionally optional. The imports of crude rubber into the United States have increased from 23,672,000 lbs. in 1884 to 61,889,000 lbs. in 1904. The value of imported rubber and kindred materials has advanced in even greater ratio. The total valuation of the imports of crude rubber, gutta percha and gutta joolatong in 1884 was only a little over 10 million dollars. In 1904 it reached nearly 44½ million dollars. Prices of rubber of late have gone soaring and best Para rubber at the present time is quoted in New York at $1.26 to $1.28 per lb. A large factor in the increased consumption and price has without doubt been the adoption of rubber tires for bicycles, automobiles and other vehicles, and the use of rubber air brake hose for freight cars. If the high prices are permanently maintained, as now seems likely, it will give a great stimulus to the cultivation of rubber on tropical plantations.

We have received a copy of the report of the Decimal Association of Great Britain dated Jan. 18, 1905, from which we learn that a bill passed the House of Lords on May 17 last making compulsory the use of the metric system of weights and measures. The bill then went to the House of Commons, which thus far appears to have taken no action upon it. Opposition to the movement appears to have developed, however, and an Association has been organized for the "Defence, Standardization and Simplification of British Weights and Measures."

That Great Britain badly needs to modernize her system of weights and measures will be conceded by both the advocates and the opponents of the metric system.

While nominally England and the United States use the same systems of weights and measures, in actual practice there is a wide difference between the two countries. Here in America the decimal system has been applied to existing weights and measures and many units which did not conform to it have become obsolete. In England, however, the old-time measures still persist. Thus the ton in the United States means 2,000 lbs., except in a few isolated cases, where the old "gross ton" of 2,240 lbs. is used. In England, however, not only is the 2,240-lb. ton the ton of common use, but its parts, the "hundred weight," which weighs 112 lbs., and the "quarter," which weighs 28 lbs., are also in common use. Apparently the Englishman has become so accustomed to cumbrous units and methods of reckoning by long familiarity with pounds, shillings and pence that the advantage of decimalizing the old weights and measures and dropping needless units never occurs to him.

We think it quite probable that this explains the apparent success of metric propaganda in England, as well as the practical failure of such propaganda here. By decimalizing the old measures in this country, we have already secured the chief advantages of the metric system.

A CRITICISM OF SOME TIMBER SPECIFICATIONS.

An engineer who would never think of calling for timber of sizes which would require cutting to order, will often write a specification for timber that, if lived up to, would necessitate selecting and grading involving more work and expense than the making of special sizes. Judging from the specifications that have been sent to our office during the past year, there are few engineers who appear to know that there are any such things as commercial classifications of timber. To illustrate the point a few quotations from current specifications will serve:

> Timber for cribs shall be of a grade which is free from decay, shakes and black and unsound knots, and having edges on one face free from sap and wane, and having not more than 2 ins. depth of sap and wane on the other edges. All other timber and plank shall be sound, free from decay, loose, black or unsound knots, shakes, splits, wane or other defects, and shall be sawn straight and square to true dimensions. The cross-section area of sap shall not exceed 2 sq. ins. in any single specified cross-section of any stick or piece. In sticks of timber 7 ins. or less in least dimension the sap shall not show on more than one corner.

So far as we know, this specification is one that has never been adopted by any association of sawmills or lumber dealers, and is, therefore, purely a creation or a copy from some previous creation of some one who appears not to have consulted dealers in or manufacturers of timber.

Here is another specification equally original:

> The lumber shall be of white or yellow pine, or any other approved kind of lumber, free from all wind shakes, large knots, decayed wood, sap or any defect that will impair its strength and durability. No sap angle will be allowed.

The foregoing clause applies to timber for bridge stringers. Apparently its author was not very clear in his own mind as to what he wanted in species of wood, but he was quite sure that each stick must have no blemish of any kind nor any sap.

The next specification applies to white oak caps and waling pieces for piles.

> All timber must be of best quality, sawed straight, free from rot, shakes, rotten knots, splits, cracks, sap wood, wanes or other defects. It will be subjected to rigid inspection before acceptance, the inspection to be done after its delivery on the work.

Here again we have originality, coupled with a determination to get something better than the best. The contractor is warned against hoping for any ruling until the material has been delivered, when he may learn that the "best" that the mills will ship is largely "culls" as viewed by the inspector.

Another specification for trestle timber is as follows:

> The packing pieces will be of white oak; all braces, caps, stringers, ties and wooden guard rails will be of long-leaf southern or yellow pine. All timber is to be cut from sound live trees, and shall be sawed full size, square in section and out of wind. It shall be free from wind shakes, large or unsound knots, pitch seams, decay or any other defects which would impair its strength or durability.

Finally we come to a government specification for dock work, and the only one in a dozen at hand which recognizes the existence of standard mill specifications. It reads as follows:

> Lumber for permanent work shall be yellow pine. Yellow pine for sheet piling shall be "standard inspection," and for all other uses shall be "merchantable inspection," according to the rules adopted by the Georgia Interstate Saw Mill Association, March 15, 1904.*

There are 15 or more associations in this country which have rules governing the inspection and classification of lumber. We have at hand the printed rules of ten of these associations, the names of which are as follows:

(1) The National Hardwood Lumber Association, A. R. Vinnedge, Secretary, Chicago, Ill.

(2) The Hardwood Manufacturers' Association of the U. S., Columbus, Ohio.

(3) Southern Lumber Manufacturers' Association, Geo. K. Smith, Secretary, St. Louis, Mo.

(4) Georgia Interstate Saw Mill Association, Tifton, Ga.

(5) Gulf Coast Lumber Manufacturers' Association, Mobile, Ala.

(6) Southern Cypress Lumber Selling Co., New Orleans, La.

(7) Baltimore Lumber Exchange, Baltimore, Md.

(8) Mississippi Valley Lumbermen's Association, Minneapolis, Minn.

(9) The New York Lumber Trade Association, 18 Broadway, New York.

(10) Pacific Coast Lumber Manufacturers' Association, Victor H. Beckman, Secretary, 603-604 Lumber Exchange, Seattle, Wash.

Of these, (1) and (2) govern the inspection of hardwood, (1) being apparently the most generally used set of rules for hardwood inspection. Rules of (3), (4) and (5) govern the inspection of yellow pine, (3) being apparently the most commonly used rules for yellow pine. Rules of (6) relate only to cypress. Rules of (7) contain reprints in part of the rules of (6) for cypress, the rules of The North Carolina Pine Association of Norfolk, Va., for North Carolina pine, and the rules of the Southern Lumber & Timber Association of Savannah, Ga., for yellow pine. Rules of (8) apply to white pine, Norway pine and hemlock. Rules of (9) apply to hardwood, gum, poplar, cottonwood and cypress. Rules of (10) apply to Douglas fir and cedar.

Since the writing of this article was begun, the "New York Lumber Trade Journal" has published "Proposed Specifications on Yellow Pine for Railway Construction," which have been recommended by a joint committee of the American Railway Engineering and Maintenance of Way Association and the Southern Lumber Manufacturers' Association. There is a separate specification for each of the following bridge and trestle members: Stringers, guard rails, caps, longitudinal x-braces, sway braces, timbers and bridge ties. There are two grades of timber for each of these, namely, "Standard Heart" and "Square Edge and Sound." The specification for "Square Edge and Sound" is the same for stringers, guard rails, etc.; but the specification for "Standard Heart" varies somewhat. The following is the proposed specification for stringers:

> *Standard Heart*: Must be sawn from sound timber; may contain certain knots, shakes or other defects of such character as will not materially affect the strength or durability of the piece. Must show 85% heart on all four sides at any given point of the piece. Must show four square edges the full length.
>
> *Square Edge and Sound*: Must be sawn from sound timber; may contain knots, shakes or other defects of such character as will not materially affect the strength or durability of the piece. Must show four square edges the full length. Sap is considered no defect.

Standard Heart for Guard Rails must show one face all heart, and other three faces 75% at any given point on the piece.

Standard Heart for Caps must show 85% heart on two parallel sides, and 75% on the other sides at any given point.

Standard Heart for Longitudinal Braces, for "Timbers" and for Bridge Ties must show 66⅔% heart on all four sides at any given point of the piece.

Standard Heart for Sway Braces must show 66⅔% heart on two faces, at any given point on the piece.

There are two criticisms that we have to offer upon these proposed specifications: First, they lack definiteness; second, they involve too much hair-splitting in the matter of percentage of heart required for different members of a trestle. We must concede that many of the published specifications of the ten associations above named are also indefinite in parts, and that it is urged with much reason that timber specifications must always be more or less vague, leaving a great deal to the judgment of the inspector. On the other hand, we purpose quoting from the rules of certain associations to show that lack of definiteness

*These rules were printed in our issue of May 12, 1904.

does not characterize them all. The following quotations are from the rules of inspection of the National Hardwood Lumber Association:

As the inspection of lumber is so largely a matter of judgment, inspectors inspecting under these rules are instructed to use their best judgment, based upon these rules, taking into consideration the general run of the stock as to lengths, widths and manufacture.

The inspection must be made from the worst side of the board, except as otherwise provided by the rules.

Lumber shall be well manufactured, of good, average widths and lengths. It should be sawed plump and even thickness and have parallel edges and square ends. Tapering lumber in standard lengths is measured at the narrow end, except as otherwise specified.

All fractions of ¾-ft. or over must be counted up to the next higher figure; all fractions of less than ¾-ft. must be counted back to the next lower figure. All badly miss-sawed lumber must be thrown out by the inspector.

The standard grades of hardwood lumber are First, Second, No. 1 Common, No. 2 Common and No. 3 Common, except as otherwise specified. When Firsts and Seconds are combined as one grade there must be at least one-third of Firsts, except as otherwise specified.

Standard lengths are 6, 8, 10, 12, 14 and 16 ft., except as otherwise specified. As most lumber is handled in lengths of 10, 12, 14 and 16 ft., odd lengths, such as 9, 11 and 13 ft., must be measured back to the next even length, except as otherwise specified.

In the grade of Firsts and Seconds the lengths are from 8 to 16 ft., but there must not be over 10% of 10-ft. lengths and 5% of 8-ft. lengths, except as otherwise specified.

The standard thicknesses of lumber are ⅛, ¼, ⅜, ½, ¾, 1, 1¼, 1½, 2, 2½, 3 and 4 ins.

Standard Defects.—One knot 1¼ ins. in diameter.

Two sound knots not exceeding in extent or damage one 1¼-in. knot.

One-in. of bright sap, except as otherwise specified.

One split, not diverging more than 1 in. to the foot, and not exceeding in length in inches the surface measure of the piece in feet, except as otherwise specified.

Worm, grub, knot and rafting-pin holes, not exceeding in extent or damage one 1¼-in. knot, except as otherwise specified.

Note.—A straight split not exceeding 6 ins. in length in one end of a piece of lumber 8 ins. or over wide must not be considered a defect.

Explanations.—The term "Sound Cutting," as used in these rules, means a piece of lumber free from rot and heart shake.

Ordinary season checks are not to be considered defects, but if of so serious a character as to damage the lumber they are to be considered by the inspector.

Black stain, heart shake, rot and dote are serious defects, reducing to a grade lower than First and Second.

Wane is a beveled edge of a board or plank as sawn from an unsquared log. Inspectors must consider same in determining grades.

In the following rules all widths and lengths mentioned are inclusive.

Special Inspection.—Log run means the full run of the log with No. 3 Commons out.

Common and Better means the full run of the log with No. 2 and No. 3 Commons out. Common and Better must contain at least 50% of Firsts and Seconds.

Merchantable means the full run of the log with No. 3 Commons out, and that the No. 1 Commons and Firsts and Seconds must be measured full and the No. 2 Commons one-half.

PLAIN-SAWN RED AND WHITE OAK.

Grades: First, Second, No. 1 Common, No. 2 Common and No. 3 Common.

Widths: 3 ins. or over.

Lengths: 4 ft. or over.

Bright sap up to one-third the width of board on one side is no defect, except as otherwise specified.

Firsts.

Firsts must be 8 ins. or over wide, 10, 12, 14 or 16 ft. long and free from all defects, except in pieces 10 ins. or over wide, which may have one sound standard defect.

Seconds.

Seconds must be 6 ins. or over wide, 8 to 16 ft. long.

Pieces 8 ft. long must be 8 ins. or over wide; pieces 8 or 9 ins. wide must be clear; pieces 10 ins. or over wide may have one standard defect.

Pieces 10 ft. or over long, 6 or 7 ins. wide and free from sap may have one standard defect; pieces 8 or 9 ins. wide may have one standard defect; pieces 10 or 11 ins. wide may have two standard defects or their equivalent; pieces 12 ins. or over wide may have three standard defects or their equivalent.

No. 1 Commons.

No. 1 Commons must be 4 ins. or over wide, 6 to 16 ft. long.

Bright sap is no defect in the Common grades.

Pieces 6 ft. long, 6 to 8 ins. wide, must be clear; pieces 9 ins. or over wide may have one standard defect.

Not over 10% of 6-ft. lengths admitted in the No. 1 Common grade.

Four-in. pieces must have one face clear and two square edges; 5-in. pieces may have one standard defect.

Pieces 6 ins. or over wide, 8 or 10 ft. long, must work two-thirds clear in not over two pieces.

Pieces 6 ins. or over wide, 12 ft. or over long, must work two-thirds clear in not over three pieces.

We will not quote further, giving specifications for No. 2 Commons, No. 3 Commons and Strips, as our purpose is mainly to illustrate the general character of these specifications.

The specifications of the Southern Lumber Manufacturers' Association (adopted Jan. 20, 1904) will next be given in considerable detail, not only because of their preciseness in most respects, but because engineers and cantractors use such large quantities of yellow pine.

YELLOW PINE.

General Instructions.

(1) Yellow Pine Lumber shall be graded and classified according to the following rules and specifications as to quality, and dressed stock shall conform to the subjoined table of standard sizes—except where otherwise expressly stipulated between buyer and seller.

(2) Recognized defects in Yellow Pine are knots, knot holes, splits (either from seasoning, ring hearts or rough handling), shake, wane, red heart, pith, rot, rotten streaks, worm holes, pitch streaks, pitch pockets, torn grain, loosened grain, seasoning or kiln checks and sap stains.

Knots.

(3) Knots shall be classified as pin, standard and large, as to size; and round and spike as to form; and as sound, loose, encased, pith and rotten as to quality.

(4) A pin knot is sound and not over ½-in. in diameter.

(5) A standard knot is sound and not over 1½ ins. in diameter.

(6) A large knot is one any size over 1½ ins. in diameter.

(7) A round knot is oval or circular in form.

(8) A spike knot is one sawn in a lengthwise direction.

The mean or average diameter of knots shall be considered in applying and constructing the rules.

(9) A sound knot is one solid across its face, is as hard as the wood it is in, may be either red or black, and is so fixed by growth or position that it will retain its place in the piece.

(10) A loose knot is one not held firmly in place by growth or position.

(11) A pith knot is a sound knot with a pith hole not more than ¼-in. in diameter in the center.

(12) An encased knot is one surrounded wholly or in part by bark or pitch. Where the encasement is less than ⅛-in. in width on both sides, not exceeding one-half the circumference of the knot, it shall be considered a sound knot. (See Sections 9 and 16.)

(13) A rotton knot is one not as hard as the wood it is in.

Pitch.

(14) Pitch pockets are openings between the grain of the wood containing more or less pitch or bark, and shall be classified as small, standard and large pitch pockets.

(15) A small pitch pocket is one not over ⅛-in. wide.

A standard pitch pocket is one not over ⅓-in. wide or 3 ins. in length.

A large pitch pocket is one over ⅓-in. wide or over 3 ins. in length.

(16) A pitch pocket showing on both sides of the piece ⅛-in. or more in width shall be considered the same as a knot hole.

(17) A pitch streak is a well-defined accumulation of pitch at one point in the piece, and when not sufficient to develop a well-defined streak, or where fiber between grains is not saturated with pitch, it shall not be considered a defect.

(18) A small pitch streak shall be equivalent to not over one-twelfth the width and one-sixth the length of the piece it is in.

A standard pitch streak shall be equivalent to not over one-sixth the width and one-third of the length of the piece it is in.

Sap.

(19) Bright sap shall not be considered a defect in any of the grades provided for and described in these rules. The restriction or exclusion of bright sap constitutes a special class of material which can only be secured by special contract.

(20) Sap stain shall not be considered a defect in any of the grades of Common Lumber.

Miscellaneous.

(21) Firm red heart shall not be considered a defect in any of the grades of Common Lumber.

(22) Defects in rough stock caused by improper manufacture and drying will reduce grade, unless they can be removed in dressing such stock to standard sizes.

(23) All stock shall be inspected on the face side to determine the grade. Stock surfaced one side, the dressed surface shall be considered the face side. Stock rough or dressed two sides, the best side shall be considered the face, but the reverse side of all such stock should not be more than one grade lower.

(24) Imperfect manufacture in dressed stock, such as torn grain, loosened grain, broken knots, mismatched, insufficient tongue or groove on Flooring, Ceiling, Drop Siding, etc., shall be considered defects, and will reduce grade according as they are slight or serious in their effects on the use of the stock.

(25) Pieces of Flooring or Drop Siding with 3-16-in. or more of tongue and pieces of Ceiling with ⅛-in. or more of tongue will be admitted in any grade. Pieces of Flooring, Drop Siding or Ceiling having not less than 1-16-in. tongue will be admitted in No. 2.

(26) In all grades of Flooring, Ceiling, Drop Siding, etc., wane on the reverse side, not exceeding one-third the width and one-sixth the length of any piece, provided the wane does not extend into the tongue, nor over one-half the thickness below the groove, is admissible.

(27) Chipped grain consists in a part of the surface being chipped or broken out in small particles below the line of the cut, and as usually found should not be classed as torn grain and shall not be considered a defect.

(28) Torn grain consists in a part of the wood being torn out in dressing. It occurs around knots and curly places, and is of four distinct characters—slight, medium, heavy and deep.

Slight torn grain shall not exceed 1-32-in. in depth, medium 1-16-in., and heavy ⅛-in. Any torn grain heavier than ⅛-in. shall be termed deep.

(29) Loosened grain consists in a point of one grain being torn loose from the next grain. It occurs on the heart side of the piece, and is a serious defect, especially in Flooring.

(30) The grade of all regular stock shall be determined by the number, character and position of the defects visible in any piece. The enumerated defects herein described admissible in any grade are intended to be descriptive of the coarsest pieces such grades may contain, but the average quality of the grade should be about midway between the highest and lowest pieces allowed in the grade.

(31) Lumber and timber sawed for specific purposes must be inspected with a view to its adaptability for the use intended. Material not conforming to standard sizes, for agricultural implement companies, wagon companies, car manufacturing companies, railway companies, etc., shall be governed by special contract and inspection.

(32) The standard lengths are multiples of 2 ft., 10 to 24 ft., inclusive, for Boards, Strips, Dimensions, Joists and Timbers, except as hereinafter specified. Longer or shorter lengths than those herein specified are special. Odd and fractional lengths shall be counted as of the next higher even length.

(33) On stock width shipments of No. 1 Common and better lumber, either rough or dressed one or two sides, no piece shall be admissible that is more than ¼-in. scant on 8-in. and under; ⅜-in. scant on 10-in., or ½-in. scant on 12-in. or wider.

(34) Yellow Pine of a better grade than No. 1 Common, up to 4 ins. in width, shall be classified as to grain as Edge Grain and Flat Grain.

Edge Grain has been variously designated as rift sawn, vertical grain, quarter sawn, all being commercially synonymous terms. Edge grain stock is especially desirable for Flooring, and admits no piece in which the angle of the grain exceeds 45° from vertical at any point, thus excluding all pieces that will sliver or shell from wear. Such as will not meet these requirements shall be known as Flat Grain.

(35) All dressed stock shall be measured and sold strip count, viz.: full size of rough material necessarily used in its manufacture.

All sizes 1 in. or less in thickness shall be counted as 1 in. thick.

(36) Equivalent means equal, and in construing and applying these rules, the defects allowed, whether specified or not, are understood to be equivalent in damaging effect to those mentioned applying to stock under consideration.

(37) The foregoing general observations shall apply to and govern the application of the following rules:

These 37 general rules are followed by about 70 rules governing the inspection of yellow pine finishing, flooring, ceiling, wagon bottoms, drop siding, bevel siding, partition, casing and jambs, boards, shiplap and siding, grooved roofing, fencing, dimension, rough finishing, common timbers, etc. Of these we will quote only the rules relating to dimension and common timbers. Dimension includes 2 × 4 up to 3 × 14 ins.

(65) Inspection of Dimension is a question of strength and uniformity of size, and whatever reduces its strength in cross-section must be considered a defect to that extent. In computing the area of cross-section occupied by defects the size of the piece in the rough shall be considered.

(66) No. 1 Common Dimension will admit sound knots, none of which in 2 × 4s should be larger than 2 ins. in diameter on one or both sides of the piece, and on wider stock which do not occupy more than one-third of the cross-section at any point throughout its length if located at the edge of the piece; or more than one-half of the cross-section if located away from the edge; pith knots, or smaller or more defective knots which do not weaken the piece more than the knot aforesaid; will admit of seasoning checks, firm red heart, heart shakes that do not go

...h, wane ¾-in. deep on edge, ¼ the width, and ⅓ the ... of the piece, pitch, sap stains, pitch pockets, splits ... not exceeding in length the width of the piece, a ... A number of small worm holes well scattered, and ... other defects as do not prevent its use as substantial ...ural material.

No. 2 Common Dimension may have knots which do ...upy more than one-half of the cross-section at any ...oint if located at the edge of the piece, nor more ...wo-thirds of the cross-section if located away from ...ge; smaller, loose, hollow or rotten knots that do ...weaken the piece more than the knots aforesaid; will ... rotten streaks, shake, wane, worm holes, split not ...ceed ¼ the length of the piece, and other defects ...h do not prevent its use without waste.

No. 3 dimension will include all pieces falling below ...2 grade which are sound enough to use for cheap ...ding material, by wasting 25% of one-third of any item ... shipment, but it must not be more than ½-in. scant ...standard finished width or ⅛-in. scant in thickness.

(77) Miscut 2-in. stock which does not fall below 1⅛ ins. ... thickness or ⅛-in. scant in width from standard size ...ll be admitted in No. 2 Common, provided such pieces ...e in all other respects as good as No. 1 Common.

No. 1 Common Timbers.

Sizes, 4 × 4, 4 × 6, 6 × 6, ⅛-in. off side and edge. Surfaced four sides, ¼-in. off each side; 6 × 8 and larger S. 4 S., ¼-in. off each side surfaced.

(78) Rough Timbers, 4 × 4 and larger, shall not be more than ¼-in. scant at any point when green, and be well manufactured, with not less than three square edges, and will admit sound knots that do not occupy more than one-third the cross-section of the piece or small defective knots.

Timbers 10 × 10 in size may have a 2-in. wane on one corner, measured on faces, or its equivalent on two or more corners one-third the length of the piece. Larger sizes may have poportionately greater defects.

Shakes extending not over one-eighth of the length of the piece are admissible, and seasoning checks shall not be considered a defect.

(79) Dressed Timbers shall conform in grading to the specifications applying to rough timbers of same size.

(80) Rough Timbers, if thicker than specified thickness for dry or green stock, may be dressed to such standard thickness, and when so dressed shall be considered as rough stock.

It will be seen, even upon casual reading of the preceding rules for inspection, that it is possible to draw timber specifications far more definite than those above proposed for railway trestles. In preparing any set of specifications to be used by manufacturers and purchasers of timber, the first essential step is to define the terms used, including "standard defects." Having done this the next step is to enumerate the number and kind of permissible "standard defects" in timbers to be used for given purposes. But in the proposed specifications for trestle timbers, above quoted, neither of these steps has been taken, the whole matter of inspection being left to the judgment of the inspector as to what defects "materially affect the strength or durability of the piece." If such an indefinite specification is adopted, we predict that it will cause no end of trouble between the manufacturers and the users of trestle timbers. Every attempt upon the part of manufacturers and purchasers to agree upon some standard set of rules for inspection is to be commended. Nevertheless, we are firm in the belief that to adopt a set of indefinite specifications is not a step forward, but will result in actually retarding the movement to secure standard specifications for manufactured products used in engineering works.

LETTERS TO THE EDITOR.

Defective Column Anchorages Due to Neglect in Erection.

Sir: Referring to your editorial in issue of Feb. 2, 1905, on the necessity of giving special care in the planning and construction of the fastenings of columns to their foundations, I would say that these details are generally well taken care of by the designing engineers, and that the fault lies in not giving the erection of the steel work the proper supervision. During the past twelve years I have been called upon to make inspection and report of the condition of numerous railroad and highway bridges and viaducts, and I have found no detail of construction so greatly neglected as the setting and fastening of the anchor bolts.

In making the inspection of one large bridge and viaduct the past year, I found that 33 anchor bolts had never been put in place, although these bolts had been provided; most of them were found hidden inside of the columns and shoes, where they were probably placed to get them out of sight and avoid the drilling of masonry and setting of the bolts. In many other cases the bolts were found to be simply dropped in the holes and not set by wedges, sulphur or cement.

This work is generally left to the last by the erector, and in the hurry to get away to another job is often neglected where there is not proper supervision of the work.

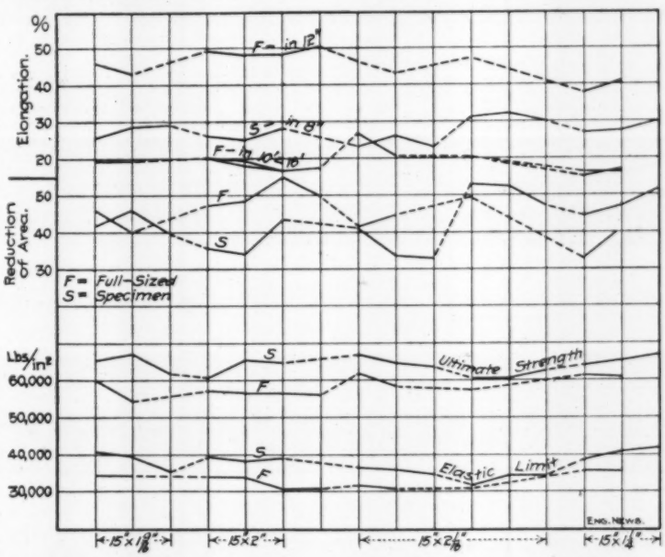
Yours respectfully,

Athens, Pa., Feb. 10, 1905. David A. Keefe.

Graphical Comparison of Full-Sized and Specimen Test of Eye-Bars.

Sir: Referring to the article on "Comparison of Full-Sized and Specimen Tests of Very Large Steel Eye-Bars," printed in your issue of Feb. 2, I enclose herewith a blue-print showing these tests as plotted by Mr. Cartlidge, Bridge Engineer of the Chicago, Burlington & Quincy Ry. The writer received a copy of these tests soon after they were made and loaned them to Mr. Cartlidge, who had them plotted.

I believe this blue-print (reproduced in the accompanying diagram) shows the comparisons in a much better and clearer way than any tabulation, and send it to you thinking other engineers may be interested in it.



GRAPHICAL SUMMARY OF COMPARATIVE FULL-SIZED AND SPECIMEN TESTS OF LARGE STEEL EYE-BARS.

Tests made at Phoenixville, Pa.; full-sized tests on Phoenix 1,200-ton hydraulic testing machine. Results reported in tabular form in Engineering News of Feb. 2, 1905.

Yours truly, Oscar J. West,
Resident Engineer, Phoenix Bridge Co.

The Rookery, Chicago, Ill., Feb. 8, 1905.

Full-Sized and Specimen Tests of Eye-Bars; Friction Error in the Phoenix Testing Machine.

Sir: Your issue of Feb. 2, 1905, is before me and I read with amazement the report on Comparison of Full-Sized and Specimen Tests of Very Large Steel Eye-Bars, in which the conclusion is stated as follows: "A fair conclusion from the values in the table would be that the full-sized eye-bars there represented have an ultimate strength 8% less, and an elastic limit 12% less, than the corresponding values of ordinary specimen test-bars."

You will note that the coefficient of error of the 1,200-ton Phoenix testing machine is nowhere stated, and it is, in your article, assumed that gage readings represent loads applied. When I was Chief Engineer of the Department of Buildings of New York City, in 1896-7, I had a 500-ton Phoenix column built for the department and then had this column tested up to 500 tons at the Watertown arsenal and at Phoenixville. I found that the Phoenix machine had 17½% friction losses at 500 tons load. At least this much must be deducted from the resistance given in your article for breaking strength. At least 12½% should be deducted from the reported values of elastic limit (sic!) (yield point). After these deductions for false indication of loads transmitted are made, where are your 8 and 12%, respectively? My statement, based on actual correct investigation by calibrated machines, is that the losses are 25% + both at yield point and at maximum strength.

Respectfully,
Gus. C. Henning.

36 Gold St., New York, N. Y., Feb. 2, 1905.

(Proof of the above letter was submitted to the Phoenix Bridge Company, and the following reply has been received from them.—Ed.)

Sir: While the friction in our large testing machine has been ascertained on several occasions, I thought it best to determine the matter again, and this was done yesterday, in the presence of the Manager and the Master Mechanic of the Phoenix Iron Company. It was found that with a pressure averaging 3 lbs. per sq. in., the piston of the machine could be forced through its entire stroke. The area of the piston being 3,000 sq. ins., the force necessary was therefore 9,000 lbs.

The elastic limit of the smallest eyebar in question, involved a pull on the machine of 660,000 lbs. The friction therefore was less than 1½% of this amount. This 9,000 lbs. total friction would apply to all tensile tests on the machine.

Yours Truly,
John Sterling Deans,
Chief Engineer, Phoenix Bridge Company.

Phoenixville, Pa., Feb. 7, 1905.

(A description with drawings of the Phoenix 1,200-ton hydraulic testing machine was given in Engineering News of Jan. 10, 1891, p. 42. As clearly shown there, the packing of both piston and piston-rods (four) is flax instead of cup-leather. This style of packing was adopted after its successful use in the Athens testing machine, the intention being to avoid the great increase of friction with increasing load, as would occur with leather packing. Both the Athens and the Phoenix machine measure the load directly by the applied water pressure, instead of actually weighing the load at the tail-block as is done for instance in the large Martens machine at the Berlin Royal Testing Laboratory.—Ed.)

The Plea of an Engineer's Wife.

Sir: As the wife of an engineer long a subscriber to your paper, I ask your permission to voice a protest and call your attention to a serious omission in your paper.

For years Engineering News has been the one necessity of our home. When in my younger, vainer and less prosperous days, I felt that some personal or household need required a five-dollar bill, I too often found that my year-before-last bonnet must still be worn with what grace I could muster, while our little extra found its way to your office in return for the paper. Of course I realized that the good received was worth while, in the larger sense; but what is the use of being credited with feminine frailties of vanity and short-sightedness in business, without occasionally using this same credit as a screen wherewith to shield one's self?

I resented—I admit it—the lack of visible return for my share of the sacrifice. I wanted theater tickets sometimes, or a tiny over-Sunday holiday or even some other magazine or publication, but those things we never could afford, while Engineering News kept on filling our mail-box, week by week to my despair.

I finally tried reading it, only to be driven to acknowledge that, to me, it was almost entirely hopeless jargon; and I felt too much sympathy with my own dear tired engineer to insist upon constant explanation. I found the gory legend on the front cover but too true and felt personally grieved as I read, "We can pump your limit" and more than disappointed by another which seemed at first sight full of hope, "If you want *anything*, follow pages 26 to 32 each week."

Like all women, from Mother Eve to Mother Hubbard, I take a normal interest in my household as well as my personal affairs, and I was cheered the other week with the hope that at last I had found something of practical service to the distaff side of the house. I found an editorial "Concerning Re-section in Plane Table Work." I ask you: doesn't that sound useful? There was another on "Plates and Sheets" and one on "A Vacuum Cleaning System." "Clam-shell and Orange Peel Buckets," although curiously worded, from my standpoint, sounded as if it might be a new salad, served in shell buckets or dishes. Then with the sartorial desire uppermost, I studied "Plugs and Feathers," not that I knew what "Plugs" were save as to a general feeling that it was the slang expression for high hats and you can imagine how aghast I stood before the idea of a feather trimmed "plug hat." Yet that at least might serve a useful purpose, while I questioned the utility of "Trimming Rock Slopes" or the medical value of an "Improved Diaphragm Pump."

Being so discouraged I was grateful for the slightest comfort for my feminine soul, and I wondered sadly if "City-Wastes Disposal Co." was a charge against the thriftiness of our civic government or a misspelled bargain sale; and what are "anti-creeping clamps" and what are "wind stresses in knee-braced mill buildings"? While I realize that there are many of us, wives as well as engineers, who, worn almost out in the service, and crippled with rheumatism, would greet with enthusiasm "Flexible Joints," still why does any one want "More information about the Volume of a Perch"? Is the "Compressed air plant" a rarer flower than the common air plant or orchid, and is it raised in that wonderful conservatory which advertises as "Manufacturing Plants"? When I looked at the "Society Proceedings" they were all concerning, and for, men! and under "Bridges" nothing was intelligible even with a fairly good knowledge of Elwell; and when I read "Blocking" with some idea that it might concern my adversary's suit, I was again disappointed.

The charm of Tennyson's "The splendor falls on castle walls" has been forever spoiled for me by a line in one of your editorials, which in its absolute lack of sense out-Brownings Browning, and yet haunts the memory like the sweeter line it parodies:

"The top cross struts between the towers."

And now having protested over much, I ask you to consider whether we women should not have some showing in Engineering News? One column is not much to beg and there are bits of experience which might be of use to others, so often similarly placed in the profession. We older women could help the younger wives perhaps, by giving them words of encouragement and suggestions how to so manage as to eke out what are too often, alas, very limited means. They could no less help us with an occasional experience seen with the eyes of beautiful, hopeful youth and recall to us our own enthusiasms and beliefs.

If I remember rightly, the cry of our colonies in the pre-revolutionary days was "No taxation without representation." Gentlemen, I ask you, can you bear to feel that we, the engineer's wives, are giving our share of the money for your paper and are receiving, personally, nothing in return? Sincerely,

"Engineer's Wife."

New York City, Feb. 3, 1905.

(We extend our sympathy to "Engineer's Wife" to such an extent that we print her letter in full, notwithstanding the fact that she failed to sign her real name, wherefore, according to the universal rule of journalism, the letter is actually entitled to no better fate than the waste basket. As to her petition: if it shall appear that the feminine members of engineers' families have interests apart from those of their sex in general, we may be led to consider favorably the granting of space in which to set forth such interests. So far as our limited field of observation extends, however, the wives and the sisters and the cousins and the aunts of engineers closely resemble the female relatives of men who are not engineers. There are a considerable number of excellent periodicals which treat of fashions, household economy, the nursery, society, etc., and are devoted solely to the special interests of women. We have yet to hear that any of these periodicals have yielded space for a "man's column," so that the head of the family may receive some small return for the personal sacrifices he makes to raise the money for his wife's yearly subscription. In fact, we doubt whether any man would have the temerity to ask such a boon.

By the same process that our correspondent shows how her husband's indulgence in Engineering News has deprived her of new bonnets, theater parties, excursions to the mountains or sea shore, all the current literary magazines and the latest historical novels, we are prepared to prove that in full many a household the self-sacrificing bread-winner of the family denies himself his after-dinner cigar, drinks beer instead of champagne, and is posted as a delinquent at his club, all on account of his wife's subscription to that feminine vade-mecum edited by the famous Bok.

Unless our correspondent will see that fair space is accorded to the masculine sex in the columns of women's journals, we shall not see our way clear in Engineering News to discuss any other kind of pattern-making than that relating to the foundry or to accept contributions on the cutting of any other teeth than those of gear wheels.—Ed.)

A Problem in Railway Location.

Sir: A few years ago when looking over a located line on railway work, I found it possible to change the line for a short distance, thereby cutting out several curves and shortening the line without materially altering the grade. The situation was approximately as indicated in Fig. 1.

The located line may be represented by the lines A, B, C, D, E, F and G; and it was desired to continue the curve A, B, to such a point that a tangent to the curve from that point would pass through the point F.

In looking up the problem that would apply here, in the field book used by the party (Shunk's) I found that it was expected that the curve should be run to such a point that a line from point F would cut the curve at two points, as B and J, and that the lines B J and J F were to be measured, also the angle J F E.

Inasmuch as point F was about 3,000 ft. from B, and very thick cedar and fir timber on swampy land lay between, it meant much time that could not well be spared, as well as considerable expense. Other field books gave practically the same solution which, of course, would not answer our purpose. The work was plotted as best we could with the appliances at hand and the central angle B O I scaled. Fortunately, the work proved sufficiently accurate for the purpose. It could hardly be expected that good results would follow this method every time.

My attention was again called to a similar problem in which the methods given in the field books at hand were as impracticable as in the former case. I then began a search for a practical solution under like circumstances with the following result. Assuming a simple problem for the general solution, such as is usually given in the field books, we have the following: "To locate a tangent

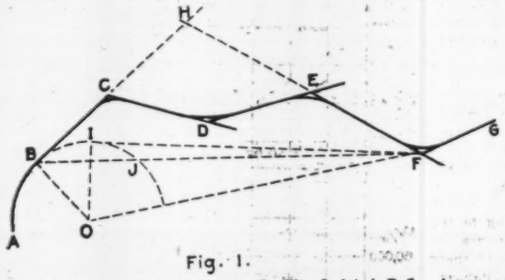


Fig. 1.

to a curve from a point without." In Fig. 2, let A B C be the curve, which has been run to the point B, and let P be the point through which a tangent is to pass. Run and measure any convenient tangent as B D to such a point D as can be seen from point P. Measure angle at D also line D P.

In the triangle B D P, sides B D and D P, also angle at D, are known, and side B P and angle N can be computed. In triangle B P O, sides B P and B O (B O = radius) also angle M (right angle minus angle N) are now known, and side O P and angle L can be computed. In triangle C O P, sides O P and C O (C O = radius) and angle O C P (O C P = right angle) are now known, and angle Y and side C P can be computed.

Angle L — angle Y = angle X = required central angle for extension of curve to tangent point C.

Of course it is not necessary that a point D be found which can be seen from P. Referring to the example from the field (Fig. 1), the tangents B C and F E can be extended (by calculations) forming the triangle B H F which corresponds to B D P in the simple problem.

Respectfully yours, Chas. C. Stowell, C. E.

Rockford, Ill., Jan. 23, 1905.

The Effect of Clay in Sand on Lean Mortar Mixtures.

Sir: In your editorial regarding sand specifications, in the issue of Feb. 2, you raise a query as to the effect of clay on lean mortars.

This leads me to offer a postscript to the excellent article in the same issue, by Mr. J. C. Hain, whose results find a parallel in many particulars, in the records of this office, where I have supervised tests of three brands of Portland and three brands of natural cement, with standardized clean lake sand; of the same sand in which from 3% to 25% of clay had been mixed; and of dirty bank sand from each of two very different pits: in 1 to 2, 1 to 3 and 1 to 5 mortars.

The results are fairly uniform in several hundred briquettes broken, except in the case given below, where an excess of water was used, and with Louisville natural, which in every case gave its highest results with clean sand.

As 1 to 3 mortars were so fully treated in Mr. Hain's article, I merely give a few results with 1 to 5 mortars, mainly taken from my report for 1903 to W. B. Patton, then City Engineer.

The sand used was standardized Lake Superior "clean, sharp sand." The clay was red, dried, pulverized and thoroughly incorporated with the sand. The results here given are averages of four and eight breaks each.

1 to 5 mortar, standardized sand, Atlas cement:

Age.		% clay in the sand.
7 days.	28 days.	
135	211	none
172	279	8
176	284	13½
153	220	21

1 to 5 mortar, Universal cement:

Age.				
7 days.	28 days.	3 mos.	1 yr.	
140	176	252	310	Clean sand.
203	301	402		10% clay in sand.

The following were made with an excess of water, and too soft to ram.

1 to 5 mortar, Universal cement:

Age.				
7 days.	28 days.	3 mos.	1 yr.	
104	180	203	251	Clean sand.
90	161	250	317	15% clay.

Very truly yours,

Edward K. Coe, C. E.,
Field Asst. City Engineer.

Duluth, Minn., Feb. 8, 1905.

PLANNING A MODERN RACE TRACK: BELMONT PARK AT QUEENS, L. I.

During the past two years a large new horse-racing park has been under construction in the immediate vicinity of New York City, adding one to the half dozen tracks already existing in the metropolitan district. The new park, called Belmont Park, is located near the village of Queens, Long Island, about fourteen miles east of Brooklyn Bridge. It has been designed to give accommodation to large crowds of spectators, to

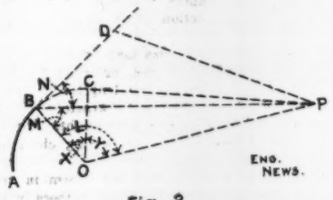


Fig. 2.

have ample and varied track facilities, and to quarter an exceptional number of horses. The planning of this park, which has been done by Mr. C. W. Leavitt, Landscape Engineer, of 15 Cortlandt St., New York, involved numerous problems of design which may be of general interest. Through the courtesy of the engineer we are enabled to present here some illustrations of the results of the work, with a brief account of the principal conditions governing the design.

The plot of ground on which Belmont Park was laid out covers an area of about 400 acres north of the Hempstead Turnpike, and about 160 acres south of that road. It was desired to lay out on this land a large modern racing park, having especially ample track facilities, and able to seat about 15,000 spectators. A large track, with an auxiliary straightaway stretch, and, if possible, a system of two tracks tangent to each other, was desired. In the latter arrangement there would be the best facilities for working horses, and there would also be opportunity for running combination races on the two tracks.

The conditions of the topography which affected the problem were in brief as follows: At the southwestern corner of the lot, just north of the turnpike, was an existing country place, the Mannice estate, completely laid out and with beautiful grounds. It was desired to preserve this as far as possible, and utilize it as a background for the club house and viewing stands. The entire land was practically level, the maximum difference in elevation in that part north of the turnpike being only about 15 ft., while the entire area on which the tracks were subsequently laid out had a maximum difference of less than 10 ft. The shape of the lot (which may be seen on the general plan, Fig. 1) had an important bearing on the work, in limiting the possible location and arrangement of the tracks.

The determining factors in the preliminary solution of a problem like the present design are the tracks and viewing stands. The stands in particular are important. They must be located immediately along the track near to the finish line, facing the track, and should at the same time command a view of the entire course; they must face as nearly as possible away from the sun, which generally restricts their orientation within a small arc in the neighborhood of northeast; and they must be so placed with reference to the

avenues of approach (railways, roads, etc.), as to facilitate the gathering and leaving of the crowds of spectators. In addition, the stands must meet numerous requirements as to interior arrangement and construction, but these do not bear directly on the layout of the grounds generally.

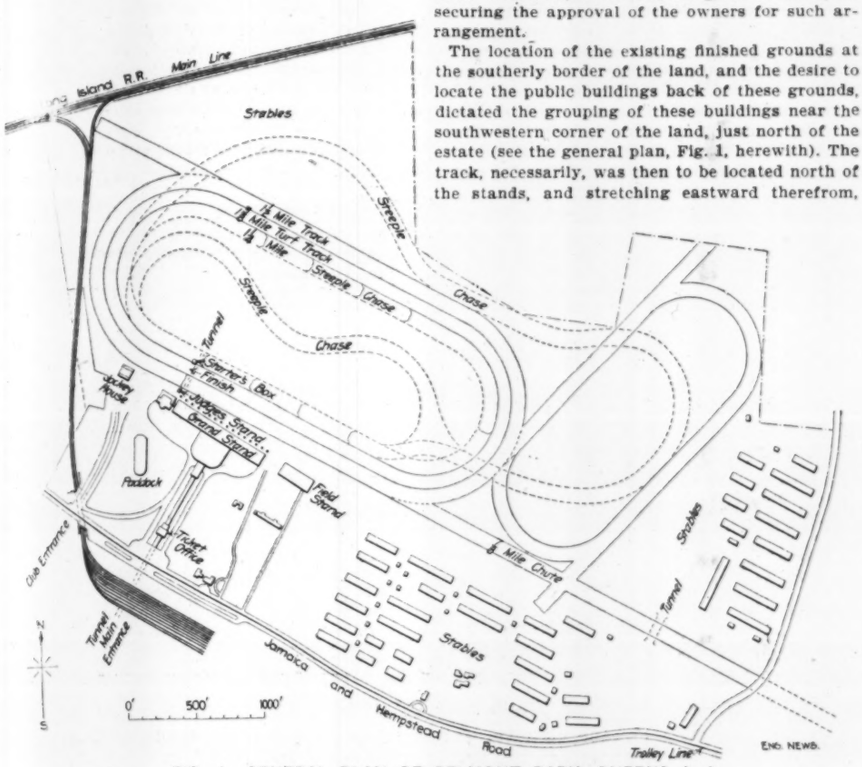


FIG. 1. GENERAL PLAN OF BELMONT PARK, QUEENS, L. I.
C. W. Leavitt, Jr., New York City, Engineer.

As to the track itself: Race courses have been built in a variety of forms; most common is the parallelogram form with semicircular ends, but there are also many elliptical, kite-shaped and other tracks. The first named form is, however, so generally used that it may, for present purposes, be called the standard form of track. The almost universal method of proportioning such a track is to make the straight sides and the curves each one-fourth the total length of the course, which means that the ratio of width to length of the course (measured between running lines, 3 ft. from the inner rail) is 1 to 2.57. In width, horse-racing tracks vary much more than in shape; widths from below 40 ft. to as much as 100 ft. may be found. The most important point of the track, the finish line, is in tracks of the "standard" form always located near the entrance to one of the curves, giving a long home stretch and giving to the spectators (the viewing stands being grouped along this stretch, as near as possible to the finish line) an excellent view of the final contest of the race. Thus, as the relative placing of race course and stands is subject to little variation, the location of either will largely determine the complete layout.

It is to be remarked that practically all race tracks in this country, whether for horse racing or other speed contests, are arranged for left-handed (anti-clockwise) running. The practice cannot, however, be said to be absolutely fixed, and as the location in the present case made the right-handed arrangement preferable, indeed almost inevitable, there was no great difficulty in securing the approval of the owners for such arrangement.

The location of the existing finished grounds at the southerly border of the land, and the desire to locate the public buildings back of these grounds, dictated the grouping of these buildings near the southwestern corner of the land, just north of the estate (see the general plan, Fig. 1, herewith). The track, necessarily, was then to be located north of the stands, and stretching eastward therefrom, thus leaving the stands, and hence the finish line, at the westerly end of the home stretch. As explained, this involved right-handed running on the track, which is contrary to usual American practice. But as one of the earliest large race tracks in this country, the Monmouth Beach track (Monmouth Beach, N. J.), was right-handed, as are also most or all European tracks, the departure is by no means an innovation in racing. In the present case the change is of less concern, because the main track (or system of tracks) is of such size as to give curves of unusually long radius, and, further, because of the fact that the group of tracks makes it possible to arrange most of the races to have only 180° to 270° curvature. The engineer's conclusion, that the local conditions dictated a right-handed track, being approved by the owners of the property, the plans were further developed on this basis, as is exhibited by the general map, Fig. 1.

The main track, facing the club house and public stands, is virtually a triple track, whose outer course is 1½ miles, a course immediately within this 1⅜ miles, and the inner course 1¼ miles, each of the three being 100 ft. wide on the stretches and slightly less on the curves. A secondary track, tangent to the outer course of the main track, is laid out directly adjoining to the eastward; this track is a single course of one mile. The one-mile track, as well as the outer course of the triple track, is of the form above called "standard." The two inner courses of the triple track are parallel to the outer one, and, therefore, depart slightly from the "standard" proportion. Winding at random through the area covered by this system of tracks is a double steeplechase course, both of whose circuits merge into the home stretch of the inner course of the main track.

In connection with the tracks there are several supplementary tangent straightaways or "chutes," which greatly amplify the possibilities of the combined tracks as to diversity of races. Thus, at the northwest corner of the main track, just opposite the club house, the back stretch is extended in a tangent westward, giving an excellent starting point for a 1⅝-mile run to the finish with only 180° of curvature, and that curvature of radius corresponding to a 1½-mile track. Again, the home stretch of the main track is extended eastward to the extreme eastern boundary of the land, and offers a straightaway course of ⅞-mile from a suitable starting line at its eastern end to the regular finish line. The secondary or one-mile track lies immediately north of this straightaway. The eastern stretch of this track is extended southward a short distance, so as to give a straightaway start for left-handed running around this track. A special race of 1⅜ miles will be made from this point around the north curve of the one-mile track, swinging into the main outer track and finishing at the regular finish line. At the points of junction between the main track and secondary track and the various straightaways, temporary barriers will be used at all times other than when a race is to be run over the two adjacent courses in combination.

A large race track requires a considerable number of buildings. In general, there are required: First, a viewing stand. Modern large race-track viewing stands are generally equipped with dining-rooms, etc., which may usually be disposed in the space underneath the seat banks. Generally, the viewing stand is divided into two portions, which have separate paths of access; the one nearer to the finish line is then called the grand stand, while the one less favorably located, and to which a lower price of admission is charged, is called the field stand. Second, a club house, offering viewing and social accommodations for the members of the racing association and their guests. Third, a saddling paddock, which should be near to the track and easily accessible from club house and grand stand. Fourth, a jockeys' building, containing dressing rooms, etc., for the jockeys, which may also contain offices and other administrative rooms. Fifth, a judges' stand and a timers' stand. Sixth, stables; these, of course, must be very extensive in the case of a large track, as is indicated also by the general plan (Fig. 1).

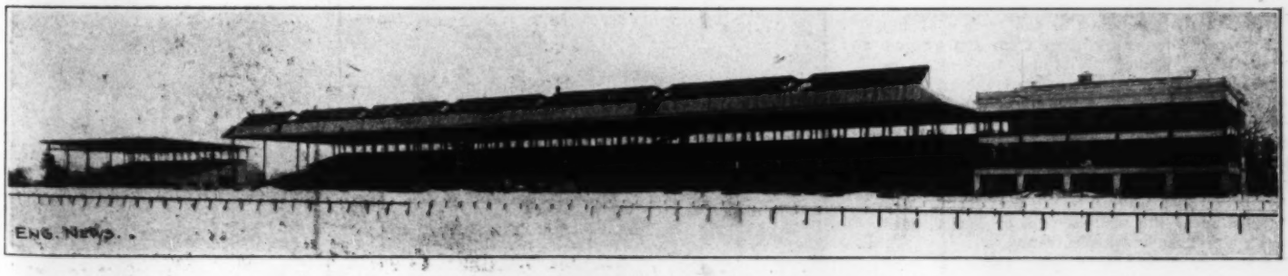


FIG. 2. GENERAL VIEW OF PUBLIC BUILDINGS AT BELMONT PARK.
(View looking South.)

The general grouping of these various buildings in the case of the Belmont Park track followed more or less directly from the conditions already outlined. The club house and the grand stand are located in the best practicable position for viewing the race; the club house lies just west of (beyond) the finish line, and the grand stand stretches eastward from that line. To the rear and between these two buildings is the saddling paddock, and about the same distance from the club house, but westward, lies the jockey house.

The field stand, east of the grand stand, is of much smaller capacity, seating but 3,750 persons, against 12,000-14,000 in the grand stand. It is separated from the grand stand by a high wire-mesh fence, designed to be as thoroughly "man-proof" as possible. The stables, whose location is more or less arbitrary, are grouped in various

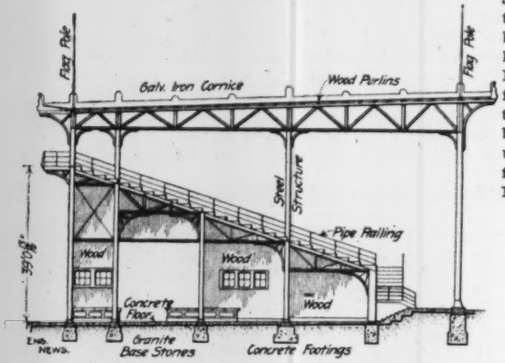


Fig. 5. Transverse Section of Field Stand, Belmont Park.

parts of the grounds not required for other purposes.

The design of the main buildings and their surroundings was influenced by the nature and arrangement of the means of access to the grounds. Belmont Park lies close to the main line of the Long Island R. R., and is served by a spur from that railway. The best available location for a terminal is just south of the Hempstead road. It is possible here to develop sufficient terminal trackage to handle the traffic to and from the park without running empty trains back over the line in either direction. A trolley line from Jamaica, connecting with the street and elevated railways of Brooklyn, runs along the Hempstead Turnpike. By developing several loops along this line, with suitable discharging platforms, it will be possible to handle a very great density of traffic on the trolley line.

From the passenger platforms on the south side of the road a wide subway approach passes under the road and thence slopes upward gradually, to a walk extending to the middle of the grand stand. This walk rises from the ground level as it approaches the stand, and enters on the mezzanine floor of a rearward extension, or rotunda, which feature of the stand will be clear from later description. A separate entrance some distance to the west permits carriages to reach the club house or a carriage stand adjacent to the grand stand. Farther to the east is another separate entrance leading to the field stand. The main portion of the visitors will, of course, pass through the grand-stand entrance.

The subway approach crossing under the road is a concrete tunnel 23 ft. wide in the clear and varying in height from 7½ to 20 ft., as its grade rises from the lower end. The tunnel is roofed with concrete arches between 20-in. I-beams spaced 3 ft. 6 ins. The entrance from the railway is at the lower end of the tunnel. Entrance from the trolley loops is by stairways leading down from the sidewalk south of the road to a walk formed on a bench or shelf in the tunnel which merges into the floor as the latter rises to the same level.

DRAINAGE.—The soil at Belmont Park is very light and sandy and allows surface water to drain off very rapidly. On this account little provision was required to be made for drainage. The general storm water collected by the drains of the western portion is collected in a 15-in. sewer, discharging south of the Hempstead Turnpike into a 24-in. sewer of the Long Island Railroad. The eastern portion of the land is drained to a 36-in. trunk discharged into a ravine southeast of the grounds. The sanitary sewage from the various buildings is collected by a separate system, and is led to a purification plant about half a mile south. Here a screen and settling chamber, a 20 x 10 x 4 ft. septic tank, a dosing chamber 15 x 10 ft., and two intermittent filters are provided. The filter beds, 30 x 15 ft., carry 4 ft. depth of sand, and are underdrained to a ditch which carries off the purified effluent. The beds are supplied from two Miller siphons in the dosing chamber, which will automatically alternate in their operation and thereby bring the two beds into service alternately.

TRACK CONSTRUCTION.—As already noted, the three tracks in the main group are 100 ft. wide on stretches. The outer one of the three narrows to 60 ft. on the curves, while the two inner tracks have their full width of 100 ft. between fences, though only the inner 60 ft. of this width is banked and finished to serve as track, the remainder forming the back slope to the ditch inside the next outer track. On the straightaways all tracks are crowned one foot transversely. Just outside the rail, either side of the track, is formed a gutter 5 ft. wide and 1 ft. deep. Catchbasins are placed in this gutter every 300 ft., and pipe drains connecting these catchbasins carry their water to the main storm sewer. No further drainage of the tracks is required under the conditions there met.

The tracks are banked about 3 ft. on the turns giving on the width of 60 ft. an *average* slope transversely of about 5%; however, the surface in cross-section is parabolic, with steepest slope at the inside (next the pole), thus providing the proper superelevation for the various degrees of curvature.

In constructing the tracks, the ground was excavated to subgrade about 1 ft. below the finished surface of the track. The filling over the subgrade was of the best portions of top soil available on the grounds, and the final surfacing was done with screened top soil. The final filling was rolled repeatedly, roughened up by harrows and re-rolled, so as to make an elastic and firm bed. The steeplechase courses are not finished in this manner, but are left more or less in their natural condition and are seeded.

VIEWING STANDS.—As may be seen from the general plan, Fig. 1, the grand stand is not quite parallel to the track, but is sloped back slightly, eastward, so as to afford a better view of the entire home stretch from the club house and the western end of the grand stand. The field stand does not follow this slope, but is set parallel to the track, which gives it a better view of the finish.

Transverse sections of grand stand and field stand, respectively, are shown in Fig. 3 and Fig.

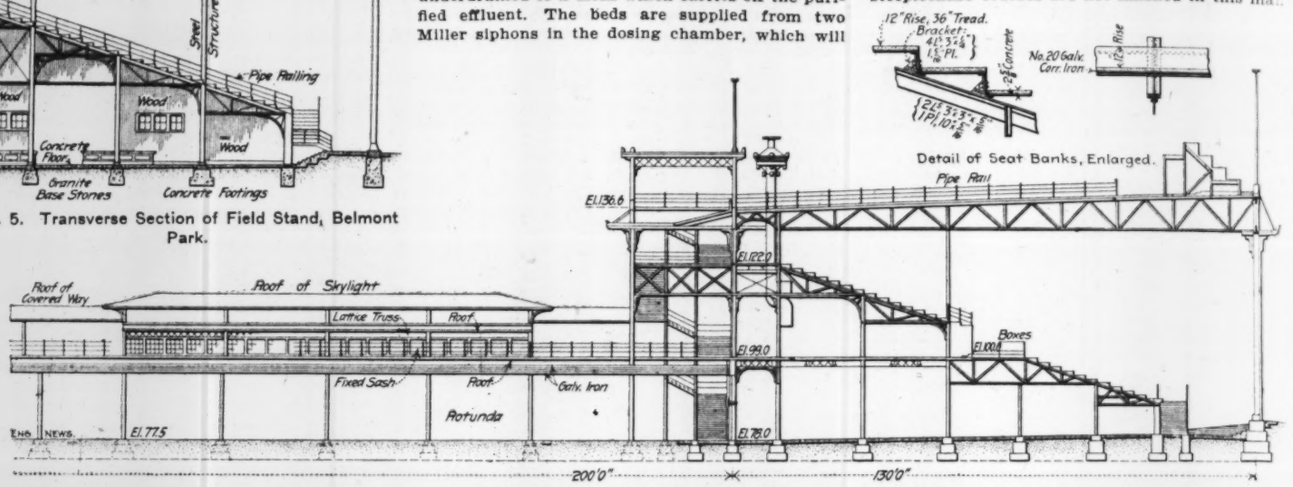


FIG. 3. TRANSVERSE SECTION OF GRAND STAND, BELMONT PARK.

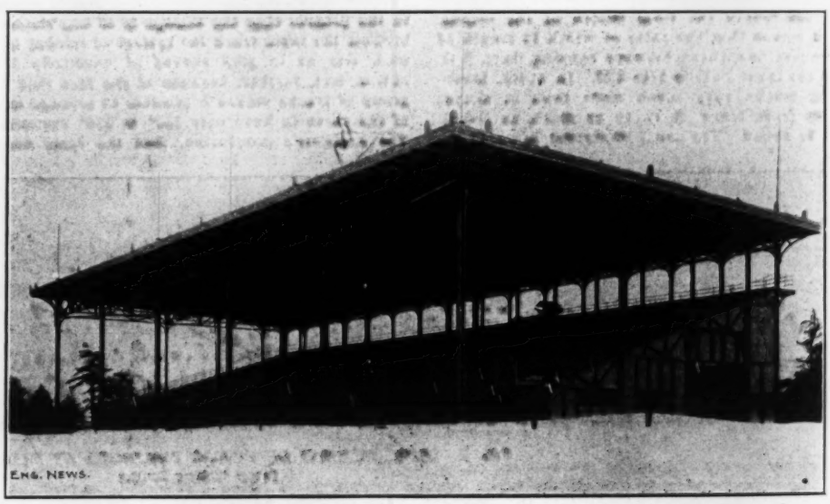


FIG. 6. VIEW OF FIELD STAND, BELMONT PARK.

5. In arrangement of framing and in structural details the two buildings are closely similar, but in general planning and subdivision the grand stand is much the more elaborate, for which reason the following matter deals mainly with this building.

The general problem in designing the grand stand was to provide not only as great a seating

capacity as practicable, but also to arrange the various required conveniences in the best manner. It is necessary, in structures of this sort, to provide a general concourse, or in some cases a betting ring; ample dining, restaurant and bar facilities must be provided, and these must be arranged so as to leave clear the means of approach and exit. The usual plan of disposing all accessory rooms on the ground floor underneath the seating banks is open to numerous objections, to obviate which in the present case a mezzanine floor was adopted. It was possible to elaborate the design in such a manner that the mezzanine floor serves as a central feature, not only of the dining and other accessory arrangements, but also of the means for approach and exit.

Structurally, the grand stand was required to furnish secure and convenient seating facilities for twelve to thirteen thousand persons, and to afford a shelter to the entire number against both vertical rain and driving rain. For this latter purpose the roof of the structure is extended some 31 ft. beyond the front edge of the stand itself. A line of columns supports the roof at the front, but these are spaced so widely as to offer little obstruction to a free view from any point of the stand. In its structural features this grand stand somewhat resembles the stand of the Empire City track at Yonkers, N. Y., which, in fact, was planned by the engineer of the Belmont Park track.

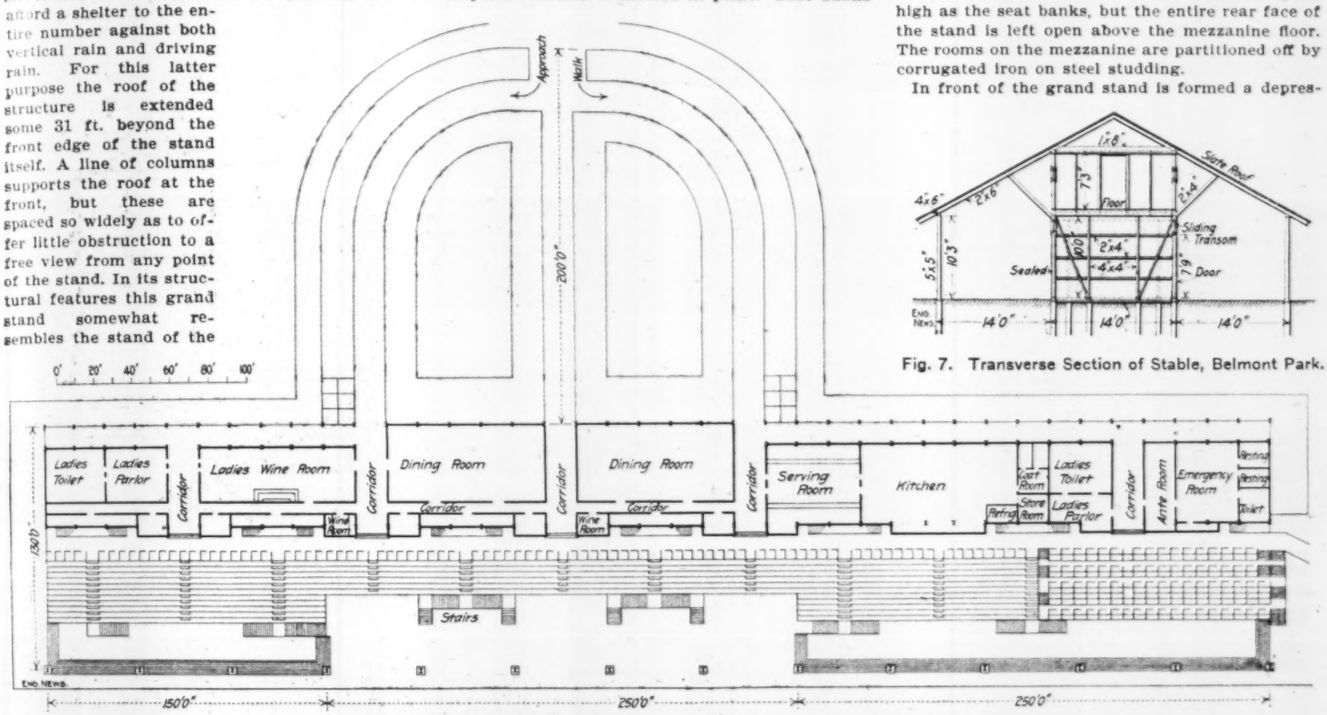


FIG. 4. PLAN OF MEZZANINE FLOOR OF GRAND STAND, BELMONT PARK.

Fig. 4, giving a diagrammatic plan of the mezzanine floor of the grand stand, and Fig. 3, a cross-section through the stand, will suffice without additional explanation to explain the general utilization of the space. The seat banks, set on a slope of 1 on 3, are supported on trusses running from front to back, which trusses are carried by columns set at relatively short intervals. The mezzanine floor meets the slope of the seat banks at about half its height; it communicates by numerous passages with a longitudinal aisle which divides the seat banks into an upper and lower half and which serves as the main passageway from which the various portions of the stand are reached. The roof columns rising through the level of the seat banks are spaced much farther apart than the columns below, it being desirable, of course, to minimize the amount of obstruction to the view from the stand. One row of columns extends longitudinally through the stand, and this, in connection with a row back of the rear seats and the forward row already referred to at the end of the overhang, supports the roof trusses. This middle row of columns is divided into bays of 16 ft. 8 ins., which is also the spacing of the trusses. The forward row of columns has a 50-ft. spacing, and a deep longitudinal truss extending over this row gives support to the forward ends of the roof trusses. The latter carry a roof of flat slope—tar and gravel on plank. At front, rear and sides, this roof drops down to the level of the bottom chord of the trusses and thus produces a hoodlike effect which architecturally is in harmony with the general open treatment of the stand. These steep edges of the roof are of corrugated steel.

The general disposition of the steel framing will, with the help of the above explanation, be readily apparent from the cross-section (Fig. 3). To this should be added one important detail, namely, the construction of the seat banks, which is shown in a detail in the same drawing. The risers of these seat banks are steel plate girders, 12 ins. deep, with one angle in each flange, and they rest directly on the trusses, forming the support for the banks. The horizontal runs, 3 ft. wide, are formed of sheets of No. 20 corrugated steel, extending from the top of one riser to the bottom of the next above, on which a 2½-in. slab of plain concrete is molded in place. Seat banks of this construction were used in the Empire City grand stand already referred to, and there proved an unqualified success, so that their adoption here was natural. Benches or settees 6 ft. long will be placed on these banks to form the seats. In the aisles the steps are subdivided by cast-iron auxiliary steps, thus giving a 6-in. rise per step.

In the middle of the grand stand there is a one-story projection to the rear, which constitutes a covered rotunda, or general concourse. This structure is roughly semi-circular in plan, 250 ft. wide by 200 ft. long, and is covered by a roof carried on five 50-ft. spans of roof trusses, resting on columns. The rotunda is open on all sides, and skylights and permanently open ventilation spaces are arranged in monitors on the roof trusses, the purpose being to provide ample light and air in this concourse for a large assemblage of people.

The roof of the rotunda is utilized as part of the approach platform leading from the railway terminal to the grand stand. The approach rises by flights of steps from the ground level as it approaches the rotunda, and continues on the roof of the rotunda in two parallel walks. These walks lead directly to the main corridor running the full length of the mezzanine floor, whence short flights of stairs give direct access to the longitudinal aisle in the seat banks. This arrangement of approaches and passages gives a very direct course for the crowds of people entering or leaving the stand, and promises to prove very successful.

The floor of the mezzanine is formed of concrete on corrugated iron (No. 20), exactly as are the horizontal runs of the seat banks. A concrete floor is also provided in the rotunda and over the entire ground surface under the stand, and is extended some 14 ft. out all around the stand as a concrete walk. The roof of the grand stand is of tar and gravel on plank throughout the structure, except for a narrow strip along the forward edge of the main roof, where five additional banks of seats of the standard construction are arranged. Two stair towers at the rear of the building, flanking the rotunda, give access to the roof seats.

The exterior of the grand stand is walled with brick up to the mezzanine floor level on all sides of the structure except the east end; the latter is walled with corrugated iron, to leave opportunity for a future extension. On the west end, facing the club house, the brick wall is carried up as high as the seat banks, but the entire rear face of the stand is left open above the mezzanine floor. The rooms on the mezzanine are partitioned off by corrugated iron on steel studding.

In front of the grand stand is formed a depression 3 ft. deep and 15 to 26 ft. wide, paved with concrete and rising by a low flight of steps to the level of the lawn beyond. This depression, from which the track can not be seen, has the object of preventing spectators standing directly in front of the building. Beyond, and falling with very flat slope to the track, is a lawn, varying in width from 100 to 150 ft. as the building inclines away from the track.

Fig. 7. Transverse Section of Stable, Belmont Park.

The field stand, 95 x 250 ft. in plan, lacks the ample dining and other accommodations of the grand stand, and hence does not require the mezzanine floor. Its ground floor is provided with bar and lunch-counters, etc. The external walls are partly sheathed with wood; a large part of the side area is left open to give ample light and air in the space below the seat-banks. Above the seat-banks, of course, the stand is open on all sides, just as in the case of the grand stand. The framing and roof arrangement is very closely similar to that of the grand stand. The roof projects forward, and is supported on widely spaced columns in the same manner, and the same seat-bank construction is used.

Architecturally, these two buildings are treated in such a way as to emphasize their light and open construction. The detailing of the roof hood, with ridges at all trusses, and the design of the walls where such occur, aims to expose and mark the lines of the framework. On the lower side of the roof the framing is wholly left open to view, carrying out the same effect.

The fence which encloses and separates the grounds of field stand, grand stand, and club

house, is built wholly of steel. It is 7 ft. high, with pipe frame covered with 1 x 1½-in. mesh wire cloth; a barbed wire is run along the top. Fences of this construction have been found to be very satisfactory, being perfectly "man-proof," and amply strong and durable.

The other two buildings in this group, club house and jockey house, may be passed over briefly, as their construction offers less of special interest here.

The club house, about 75 ft. square in ground area, is elaborate in its appointments, as suited to the purposes for which it is intended. It is subdivided about thus: entrance halls, kitchen and dining-rooms are on the first floor; a large ladies' dining-room, 36 x 75 ft., and other assembly rooms, on the second floor; café, private dining-rooms and bedrooms on the third floor; and dormitories, laundry, and other service rooms in the deck-house. A freight and a passenger elevator are provided. The walls are mainly of brick, with terra-cotta trimmings, above a cut-stone base, and the interior framing is of timber, except that cast columns and steel plate-girders are used to support the long spans of floor. Wide balconies extend around front and sides of the building; the front balcony has its floor arranged in steps 5 ft. wide, so that it may be used for dining and yet afford a full view of the track.

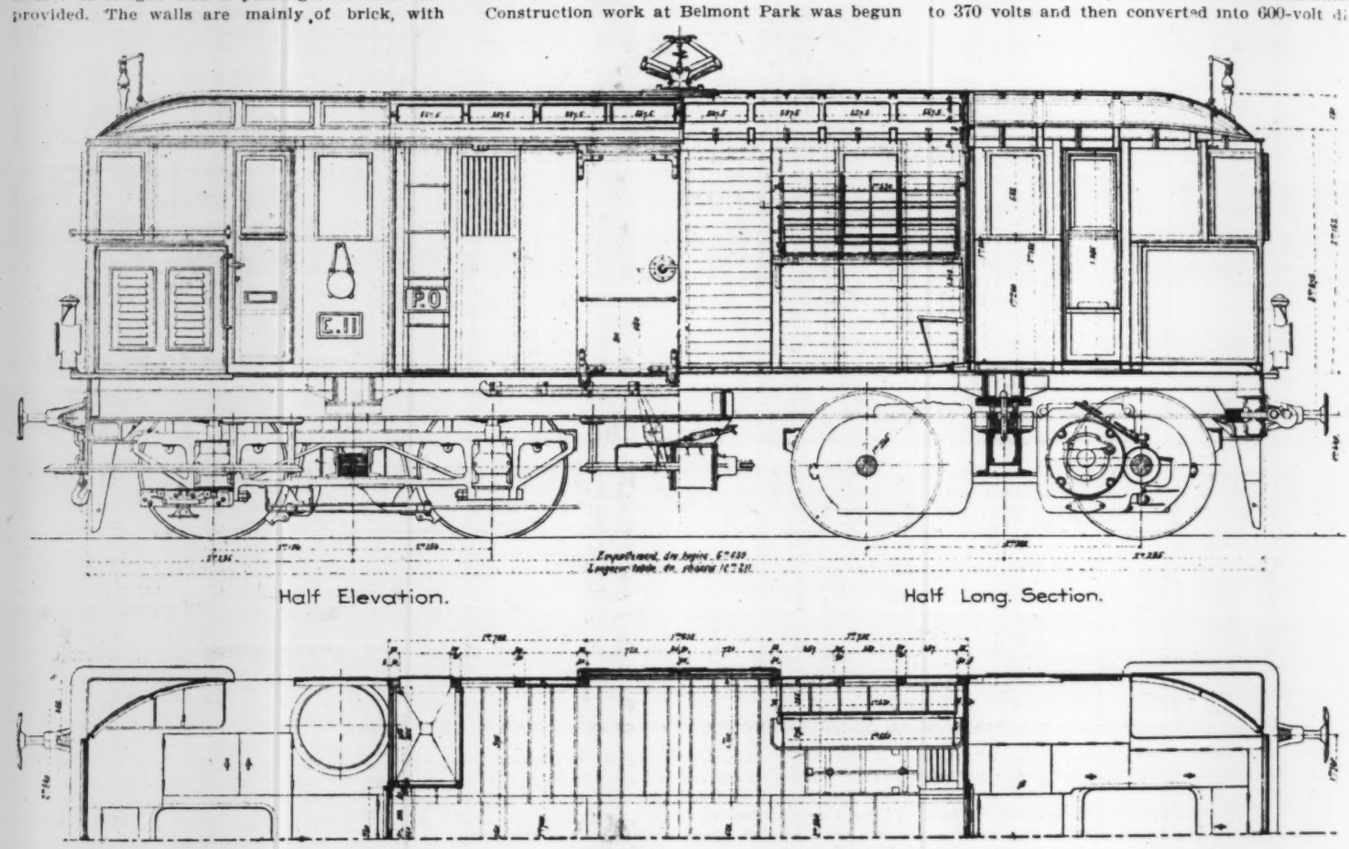
The club house appears at the right in the view Fig. 2. The long pillars shown at the front of the balconies are round cast-iron columns, 20 ins. x 31 ft., with an ornamental sheet-copper capital at the top.

The jockey house, 82 x 73 ft. in plan, is a two-story frame structure similarly provided with balconies at front and rear. The first floor contains a jockey's dressing and locker room, 42 x 45 ft., bath-rooms, a public hall (for owners and trainers), and some offices. On the second floor are administration rooms, stewards' meeting-rooms, and the like.

The saddling-paddock, to the rear of the club house, is substantially a large open shed, framed in yellow pine. It is 82 ft. x 300 ft. in plan, with semi-circular ends. Two longitudinal rows of posts, 55ft. apart, support the roof-trusses (timber). Along each side runs a row of box stalls, 18 in a row or 36 in all; the remaining floor-space is unoccupied.

The stables throughout the grounds, about 40 in number, are of uniform design, differing only in length, according to the number of stalls. Their construction is conventional; Fig. 7 gives a transverse section. The width of the stable proper is the length of a single stall, 14 ft., but a wide overhang of the roof, 14 ft. all around, affords a partially sheltered space for exercising horses. The different stables vary from 10 to 30 stalls in length, each stall being 10 x 14 ft., thus making the building 100 to 300 ft. long. The stalls are ceiled with 2-in. plank. Above is a hay-loft, and the whole is covered with a slate roof. The door of each stall is in two halves, upper and lower, and is made of hard wood.

Construction work at Belmont Park was begun in April, 1903, and at the present time is nearly completed. The general work was done by the New York Contracting & Trucking Co., and Pennell & Oliver. The steel work of grand stand and field stand was fabricated and erected by the American Bridge Co., of New York. The other buildings were constructed by Fountain & Choate. The plumbing, of which is a very large amount, was installed by John J. Welstead. The planning and designing and the supervision of the entire undertaking from the beginning was done by Mr. C. W. Leavitt, Jr., of 15 Cortlandt St., New York City.



Half Elevation. Half Long. Section.

Half Sectional Plan.

FIG. 1. ELECTRIC LOCOMOTIVE; PARIS & ORLEANS RY., FRANCE.

ELECTRIC TRACTION ON THE PARIS & ORLEANS RY. AT PARIS, FRANCE.

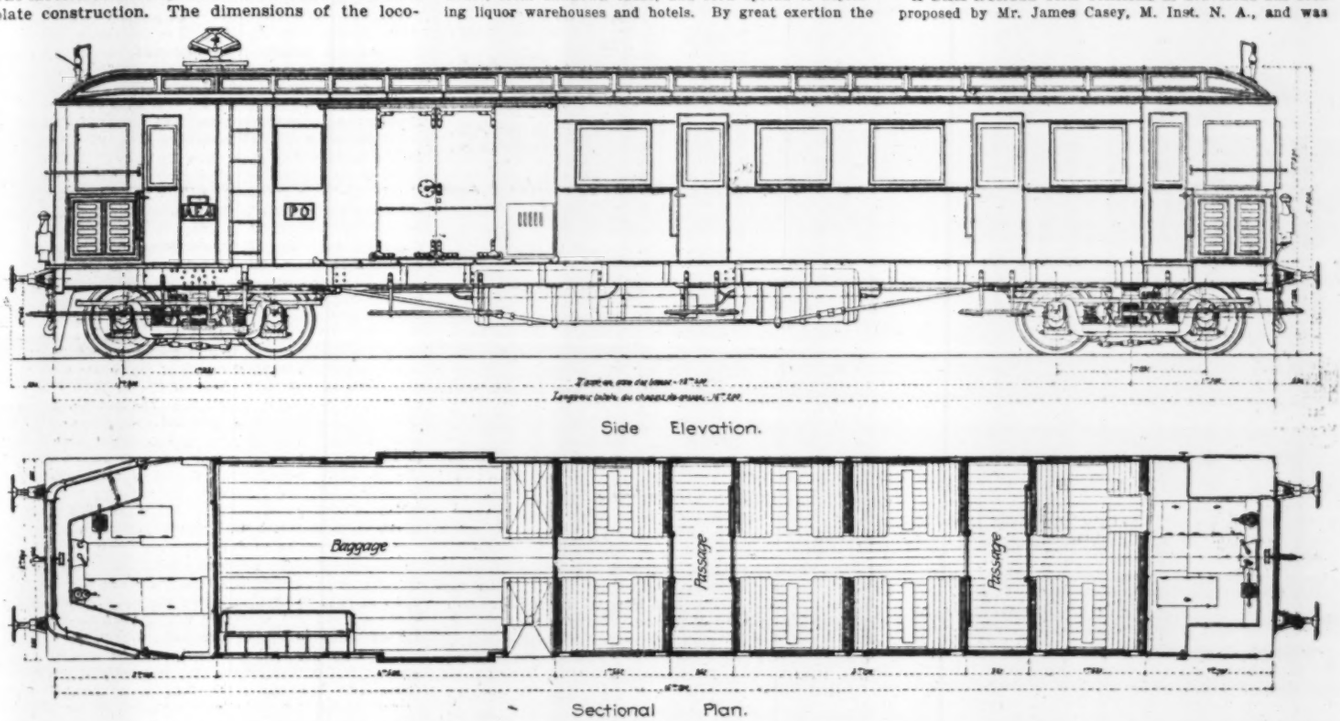
The passenger service of the Paris & Orleans Ry. at Paris, France, is notable in that the main line trains are hauled by electric locomotives on an underground line through the city to the new terminal station, and that the suburban trains are operated by electric traction. Some notes of this system and of the works involved, from an article in the "Revue Generale des Chemins de Fer," were given in our issue of Aug. 18, and we are now enabled to give some further particulars of this interesting system. The electric locomotives haul from 150 to 200 trains daily, weighing 150 tons as an average and 350 tons maximum, at a speed of 31 miles an hour, the speed being limited on account of curves of 492 ft. and 656 ft. radius. The run from the terminal to the station where the steam locomotives are attached and detached is made in 6 to 7 minutes, and the change of engines is made in three minutes as an average.

The electric traction for suburban service is in force between Paris and Juvisy, 14 miles, where the grades are easy and there are no curves of less than 2,624 ft. radius. There are four tracks, the two inner tracks being for the main line steam trains, while the outer ones are equipped with the third-rail conductor for electric traction. The power is generated at the Ivry power station, and the 5,500-volt three-phase current is transformed to 370 volts and then converted into 600-volt direct-current at two substations. The power house contains three generating units of 1,000 KW. each, driven by Dujardin four-cylinder triple-expansion Corliss engines of 1,500 HP.; the high pressure cylinder is 24¼ ins. diameter, and all the others are 42 ins. diameter; the stroke is 66 ins. The conductor is a 52-lb. T-rail with a flat bar on each side, like a continuous splice; these bars weigh 26 lbs. per yd. The rail and bars are of Siemens-Martin steel, containing 0.065% carbon and 0.433% manganese. The conductor is carried by blocks of creosoted wood on the ends of the ties. At stations the third rail is protected, and where the rail is broken at switches, etc., the ends are connected by cables laid in the ballast. Interrupters enable certain sections of the conductor to be cut out without affecting the rest of the line. These can be operated from a distance by means of wire connections to the interlocking cabins and signal cabins. Telephone connections are placed at the stations and on the poles of the block signals. The track rails are of bull-head section, and on all four tracks the joints of these rails are bonded by copper connections between the rail webs and the splice bars.

There are 11 electric locomotives, Fig. 1, each having a pair of trucks with a motor on each axle. The engines can run at 62 miles an hour without a train, or 43½ miles an hour with a train of 200 tons. The latest engines have the electric apparatus, air compressors, etc., arranged in a cabin at each end, the central portion being used as a baggage car. Besides the locomotive for trains, there are five motor cars, Fig. 2, forming two trains for local service, with one car in reserve. Each train is composed of two motor cars, with five to seven ordinary cars between them, all the motors being controlled from the head of the train. This arrangement avoids all terminal switching. The train weighs 175 tons empty, and has seats for 520 passengers. The motor cars have two trucks, with a 125-HP. motor for each axle, and the body contains a motorman's cabin, baggage compartment and passenger compartment, with seats for 32 persons. The underframe is of steel, and to provide against fire the floor is sheathed with a fireproof composition and the body is built of fireproofed wood. The motorman's compartment, however, is of steel plate construction. The dimensions of the locomotives and motor cars are as follows, and in the latter the trucks represent 12 tons of the total of 42 tons:

	Electric locomotives.		Motor cars.	
	ft.	ins.	ft.	ins.
Length over end sills	33	6	53	2
Length over all	37	4	57	0
Length of body	..	..	53	2
Width, interior	8	10	8	4
Width, exterior (extreme)	10	2	10	2
Height above rail	12	8	12	6
Truck wheelbase	7	10	6	6
Distance c. to c. of trucks	18	6	40	8
Wheels, diameter	4	0	3	4
Weight, total	53 tons.		42 tons.	

This suburban service has been in regular operation since July 1, 1904, and 70 to 80 trains daily are hauled between Paris and Juvisy; most of these do not run beyond Juvisy, and the others are taken on by steam locomotives, the traffic not being sufficient to warrant the electrical installation. The distance from the Paris terminal station (Quai d'Orsay) to Juvisy is 14 miles. With eight intermediate stops of half a minute to one minute, the trip is made in 34 to 38 minutes, or at an average speed of 28½ miles per hour. Trains making only two stops make the trip in 26 minutes, and their average speed is about 31 miles an hour. The average daily mileage is 124 miles for the electric locomotives and 155 miles for the motor cars.



Side Elevation.

Sectional Plan.

FIG. 2. ELECTRIC MOTOR CAR; PARIS & ORLEANS RY., FRANCE.

THE LARGEST PUMPING ENGINE EVER BUILT was put into operation at the Calf Pasture Pumping Station, Boston, Mass., on Feb. 14, 1905. The pump is of the Leavitt type, and has a capacity of 72,000,000 gallons per day of 24 hours. Its cylinders are 18½, 33 and 52¾ ins. by 120 ins. stroke. The two pump plungers are 60 ins. in diameter. The engine has a 36-ft. flywheel weighing 55 tons.

A DERAILMENT on the Erie R. R., near Rutherford, N. J., on Feb. 20, caused the death of one passenger and the injury of more than a score. The accident occurred on the Bergen County Cutoff, a new seven-mile line for fast trains, which extends from Rutherford to Ridgewood, N. J., and bypasses the cities of Passaic and Paterson. The train wrecked was the fastest short-distance express on the line, and carries high-speed commuter traffic from Middletown, Suffern and Tuxedo, N. Y., to New York City. It is thought that the derailment was caused by the breaking of a tender axle. The locomotive kept to the track, but all the cars were derailed and were dragged over the ties for nearly a quarter mile.

A CONFLAGRATION occurred in Indianapolis, Ind., during the evening of Feb. 20, producing a loss estimated at $1,100,000. The fire began in a wholesale millinery house, from unknown cause, and soon spread to adjoining liquor warehouses and hotels. By great exertion the fire was finally confined to the block in which it started, but of this just one-half was wholly destroyed.

AN EXPLOSION IN A SUBMARINE BOAT at Queenstown, England, on Feb. 16, 1905, killed four men and wounded several others. The accident occurred while the boat was preparing for a service exhibition. It is thought to have been caused by accidental ignition of gasoline while the gasoline tanks of the boat were being filled. In these boats power is supplied by electric motor when submerged and by gasoline engine when on the surface.

A SLAG-LADLE CAR, which is tipped by the pull of a chain, and which dispenses with the use of gearing or capstans and hand labor for tipping, has been placed on the market by the Wellman-Seaver-Morgan Co., of Cleveland, O. A steel sheave is attached to the car underneath the lip of the ladle, and a chain runs over this sheave having one end fast to the car body. To dump the car the other end of the chain is attached to the locomotive so that the pull of the locomotive tips the ladle. A train of side-tipping ladles can be tipped in series by simply attaching the chain on each car to the drawbar of the car ahead.

THE CANAL SYSTEM OF PRUSSIA is to be extended on a large scale if a bill which has been favorably recommended to the Prussian Diet becomes a law. According to Consul-General Guenther of Frankfort the canals recommended are as follows:

1. A navigable canal between the Rhine and Weser, with a connection to Hanover, and the canalization of the Lippe:

(a) A canal from the Rhine, Ruhrort, to the Dortmund-Ems canal or the vicinity of Herne (Rhine-Herne canal), inclusive of a branch canal from Datteln to Hamm; estimated cost, $17,731,000.

(b) Additional works on the Dortmund-Ems canal between Dortmund and Bevergern; estimated cost, $1,463,700.

(c) A canal from the Dortmund-Ems canal, Bevergern, to the river Weser, connecting with Hanover; branch canals to Osnabrück, Minden and Linden, construction of reservoirs in the upper parts of the river Weser and some regulation works of the Weser below Hameln; estimated cost, $28,679,000.

(d) Canalization of the Lippe or construction of branch canals from Weser to the Dortmund-Ems canal, near Datteln, and from Hamm to Lippstadt; estimated cost, $10,014,800.

(e) Improvement of the cultivation of the soil in connection with the works under items a to d, and the completed Dortmund-Ems canal; estimated cost, $1,190,000.

2. A deep waterway between Berlin and Stettin; estimated cost, $16,234,000.

3. Improvement of the waterway between the rivers Oder and Weichsel, also of the river Warthe from the river Netze to the city of Posen; estimated cost, $5,039,650.

4. The canalization of the Oder from the Glatzer Neisse to Breslau, experimental works between Breslau and Fürstenberg on the Oder, construction of reservoirs; estimated cost, $4,676,700.

The entire cost of the projects named is placed at 334,575,000 marks ($79,628,880).

A DAM ACROSS THE THAMES at Gravesend has been proposed by Mr. James Casey, M. Inst. N. A., and was made the subject of a paper by him read before the Civil and Mechanical Engineers Society on Jan. 5. The double purpose of the barrage, as Mr. Casey calls it, is to improve the navigation of the river and to afford a great harbor, "capable of holding the entire British fleet afloat." At the site of the proposed dam the river is about 2,400 ft. wide and has a depth of 45 ft. at low water. At London Bridge the river at low water is 650 ft. wide and 10 ft. deep, while at Teddington weir the low water figures are 250 ft. and 6 ft., respectively. The distance by river from Teddington to Gravesend is 46 miles, the average width of the river is about one-third mile and the fall of the bed is about 1 ft. a mile. The rise of the tide at Gravesend is from 15 ft. to 20 ft. 9 ins. at Gravesend and at London Bridge from 17 to 21 ft. The proposed dam would be of solid concrete, with granite facings, provided with four locks, two of which are to be 1,000 ft. in length, and two of 800 ft., and 100 ft. and 80 ft. in width, respectively, provided with internal steel gates dividing them into lengths of 300 ft. and 700 ft. in the case of the larger docks, and 300 ft. and 500 ft. in the case of the lesser ones. In addition to the outer gates, a number of adjustable steel sluices will also be provided, fitted at the top and bottom of the barrage, and having sufficient area to regulate the flow and necessary scour of the river bottom during ebb tide.

In the base of the barrage would be constructed a railway tunnel connecting Kent and Essex with the northern lines, which from a commercial and strategic point of view would be most valuable. At present all passengers, goods and luggage traffic to and from the Continent has to pass through London, there being no alternative route, but if this railway connection were made, the traffic passing between the Continent and the Eastern and Midland counties and Scotland would pass through without change of carriage or touching London.

The estimated cost of the dam, tunnel and auxiliary

works is about $20,000,000 and the time required to build it is placed at three years.

In support of his project Mr. Casey refers to the Charles River dam (see Eng. News, Oct. 6, 1903, and Jan. 12, 1904). This dam he wrongly says is an enterprise of the United States Government, whereas it is being built by the State of Massachusetts at the ultimate joint expense of the Metropolitan Park District and the cities of Boston and Cambridge. He makes the interesting suggestion that since the commission on Vehicular London Traffic has sought the advice of the former chief engineer of the New York subway railroad the precedent should be followed and the experts responsible for undertaking the barrage in the Charles River be invited to give their opinions and advice in the case of the Thames barrage.

Mr. Casey's paper is printed in full in the London "Surveyor" for Jan. 5, 1905.

PERSONAL.

Mr. R. W. Barwick has been elected City Engineer and Superintendent of Water-works of Cordelle, Ga.

Mr. T. C. Moorshead has been appointed Superintendent of the Litchfield & Madison Ry., with headquarters at Edwardsville, Ill.

Mr. R. H. Rogers has been appointed Master Mechanic of the New York, New Haven & Hartford R. R., at South Boston, Mass., to succeed Mr. C. J. Howe, resigned.

Mr. Samuel Whinery, M. Am. Soc. C. E., has been appointed Consulting Engineer for the Borough of Manhattan, New York City, in all matters relating to street pavements.

Mr. H. C. Bayless, hitherto Chief Draftsman of the Minneapolis, St. Paul & Sault Ste. Marie R. R. has been appointed Mechanical Engineer, with headquarters at Minneapolis, Minn.

Mr. Arthur F. Hawkes, a civil engineer formerly in the employ of Clapp & Abercrombie, Engineers, Greenfield, Mass., has accepted an appointment as Instructor in Civil Engineering at Harvard University.

Mr. Edward W. Bush and Miss Martha Wells Graves were married Feb. 7 at Hartford, Conn. Mr. Bush is a civil engineer and at present is engaged on the construction of the New Connecticut River bridge at Hartford.

Mr. R. E. Taft has been appointed Resident Engineer of the Great Northern Ry., with headquarters at West Superior, Wis. Mr. H. F. Hamilton has also been appointed Resident Engineer, with headquarters at Havre, Mont.

Mr. Herbert A. Clarke, formerly Chief Engineer of the Vandegrift Construction Co., has opened an office in the Perry Building, Philadelphia, Pa., and will devote his time to private practice, making a specialty of electric railways.

The headquarters of the Third Mississippi River District, extending from the mouth of White River to Warrenton, has been transferred from Memphis, Tenn., to Vicksburg, Miss., and from Capt. E. Van C. Lucas to Capt. G. M. Hoffman, Corps of Engineers, U. S. A.

Wm. Gerig, M. Am. Soc. C. E., who has been an Assistant Engineer with the Mississippi River Commission for a number of years, has been appointed Superintendent of Dredging Operations, on the Mississippi River, to succeed Mr. F. B. Maltby, M. Am. Soc. C. E., whose appointment as Division Engineer under the Isthmian Canal Commission, was mentioned in our issue of Feb. 2.

Mr. F. S. Cook, for 21 years Assistant Engineer in the office of the Chief Engineer of the New Croton Aqueduct, in charge of plans and designs, has been appointed Division Engineer by the Aqueduct Commission and is placed in charge of the Jerome Park Reservoir. Mr. Cook, in point of continuous service, is the oldest member of the engineering staff of the Aqueduct Commission.

Obituary.

H. J. Stanley, for 25 years City Engineer of Cincinnati, O., died Feb. 9, after an illness of several months.

Col. C. A. Nash, head of one of the largest building supply houses in the South, died suddenly of heart failure, Feb. 10, at Pinehurst, N. C.

Isaac Eppinger, Treasurer of the Eppinger & Russell Co., lumber dealers of New York City, died Feb. 5 at his home in that city, aged 78 years.

J. W. Gephart, Superintendent of the Central Railroad of Pennsylvania, died suddenly at his home in Bellefonte, Pa., Feb. 14, of apoplexy. He was 52 years old.

Rudolph J. Sherk, formerly Superintendent of Water-works of the City of Lebanon, Pa., died suddenly of heart failure, Feb. 13, at his home in Lebanon. He was 58 years old.

John G. Bogert, a retired builder, died Feb. 14, at his home in New York City. Mr. Bogert with his brother, the late Albert G. Bogert, are said to have erected the first stone and iron building put up in New York at Prince St. and Broadway. He was 70 years old.

Norton P. Otis, Chairman of the Board of Directors of the Otis Elevator Co., died Feb. 20, at his home in Yonkers, N. Y., Mr. Otis, who was born 65 years ago, was the son of Elisha G. Otis, the inventor of the Otis Elevator. Mr. Otis had been a member of Congress from the 19th New York District since 1903.

Lluz Evaresto de Costa Cebral, a manufacturer of machinery, of Rio de Janeiro, Brazil, died suddenly Feb. 16, at the Hotel Breslin, New York City. He was considered one of the wealthiest men in South America, having agencies all over the world, and was on a protracted visit in this country. The climatic changes brought on heart and lung trouble.

Capt. W. F. Van Buskirk, formerly Assistant City Engineer of Winnipeg, Man., died Jan. 15, at the residence of his father-in-law at Stratford, Ont. Mr. Van Buskirk was graduated from the civil engineering course of the Royal Military College at Kingston, Ont., winning a commission in the Imperial Army, which he did not accept. He afterwards took a course at the Massachusetts Institute of Technology, and began the practice of his profession at Woodstock, Ont. Later he was connected with the engineering work in connection with the water-works system at Orange, Mass. Previous to his becoming Assistant City Engineer of Winnipeg, he was City Engineer of Stratford, Ont., and later on was City Engineer of Rossland, B. C. He was 43 years old.

ENGINEERING SOCIETIES.

COMING MEETINGS.

ASSOCIATION OF ONTARIO LAND SURVEYORS.
Feb. 28, 1905. Annual meeting at Toronto, Ont. Secy., Killaly Gamble, Temple Bldg., Toronto.

AMERICAN RAILWAY ENGINEERING AND MAINTENANCE OF WAY ASSOCIATION.
March 21, 22, 23. Annual meeting at Chicago. Secy., L. C. Fritch, 1562 Monadnock Block, Chicago.

AMERICAN ELECTROCHEMICAL SOCIETY.
April 25 to 27. Annual meeting at Boston and Cambridge, Mass. Secy., S. S. Sadtler, 39 South 10th St., Philadelphia, Pa.

AMERICAN WATER-WORKS ASSOCIATION.
May 8 to 12, 1905. Annual meeting at West Baden, Ind. Secy., J. M. Diven, Charleston, S. C.

INTERNATIONAL RAILWAY CONGRESS.
May 3 to 15, 1905, at Washington, D. C. Secy. of American Committeee, W. F. Allen, 24 Park Place, New York.

MASTER CAR BUILDERS' ASSOCIATION.
AMERICAN RAILWAY MASTER MECHANICS' ASSOCIATION.
June 14-21, 1905. Annual meeting at Manhattan Beach, N. Y. Secy., J. W. Taylor, The Rookery, Chicago, Ill.

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.—At the next meeting, to be held at 154 West 57th St., New York City, on the evening of Feb. 24, 1905, Mr. N. M. Crawford, of Hartford, Conn., will present a paper entitled "Two-Motor vs. Four-Motor Equipments," and Mr. C. W. Ricker, of New York City, a paper on "Track Bonding."

RAILWAY SIGNAL ASSOCIATION.—The next meeting of the association takes place at the Great Northern Hotel, Chicago, Ill., at 2 p. m., March 14, 1905. The paper "Storage Batteries for Block Signals," by Mr. Reynolds, read at the last meeting, will be further discussed. The report of the committee on Standard Specifications for Mechanical Interlocking and Material for Construction Work will be open for discussion. The committee on Definitions and Nomenclature will submit a further report, which will come before the meeting for discussion. Secy., H. S. Balliet, South Bethlehem, Pa.

ENGINEERS' SOCIETY OF WESTERN PENNSYLVANIA.—At the next general meeting of the society, to be held at 410 Penn Ave., Pittsburg, Pa., on the evening of Feb. 21, Mr. H. W. Dubois will read a paper on "A Recognizance for the Platinum Metals in British Columbia." The Chemical Section will meet on Feb. 23, when Mr. H. E. Walters will read a paper entitled "Some Experiments on the Determination of Antimony in Alloys." At a meeting of the Mechanical Section on March 7, a paper on "Superheated Steam," will be presented by Mr. J. L. Moore. At the next meeting of the Structural Section, on Feb. 28, a paper on "Forestry" will be presented by Mr. O. W. Pun.

NEW YORK ELECTRICAL SOCIETY.—The February meeting of the society was held at the rooms of the Metropolitan Ry. Employees' Association, Seventh Ave. and 50th St., on the evening of Feb. 15. Mr. H. H. Vreeland, after a pointed introduction by President F. J. Sprague, spoke on "Electric Railway Operation in a Great City," wherein he dwelt largely on the requirements to be met by street railway apparatus, beyond merely operation under laboratory conditions. The importance of simplicity was forcibly brought out, and the many complications involved in the introduction of new methods or apparatus were illustrated by the transformation of the cable lines in New York City to electric conduit operation, without interrupting traffic. Mr. Vreeland spoke at length on the importance of training employees and of establishing an efficient and stable organization by proper discipline, graded pay, seniority, etc. At the close of his address the production of motormen from raw applicants was illustrated by an actual performance representing classes from the instruction school. Secy., G. H. Guy.

SOCIETY OF ENGINEERS OF EASTERN NEW YORK.—A well-attended meeting of the society was held at Troy, N. Y., on Feb. 15, 1905. During the afternoon the members visited the gun shop of Watervliet Arsenal, under the guidance of Capt. O. C. Horney, U. S. A. After a banquet at the Rensselaer Hotel, a business session was held at the Y. M. C. A. rooms. Here Capt. Horney supplemented the afternoon's excursion by a lecture on "Modern Ordnance."

The next meeting, which will be the annual business meeting, will be held on April 12, 1905. The election of officers occurs at this meeting; the nominees are: For President, Prof. Olin H. Landreth, of Schenectady (vice H. G. Hammett of Troy); Vice-President, Capt. O. C. Horney, of Troy (vice O. H. Landreth, of Schenectady); Treasurer, Mr. W. I. Slichter, of Schenectady (continued); Member Finance Committee, Mr. J. A. Kinkead of Schenectady; Member Executive Committee, Mr. F. E. Crane, of Amsterdam. Secy., A. E. Cluett.

NEW YORK RAILROAD CLUB.—At the regular monthly meeting of Friday, Feb. 17, 1905, Mr. Charles Streicher, of Butler, Pa., presented a paper on "Running Railroad Shops on the Monthly Appropriation Plan." The paper strongly urged the advantages of this plan in securing more efficient conduct of railway repair shops, by basing their operation on business principles. Statistical figures as to annual maintenance cost per locomotive and per car were cited to show that the repair department costs have increased in recent years while all other railway departmental costs have decreased. The primary source of efficiency in shop work was held to be the energy, resourcefulness and originality of the individual, workman as well as manager; the author holds that developments in shop management have not tended to promote this agent, but rather to oppose and repress it. Operating by the monthly appropriation plan, he believes, will recall it to vigor.

The discussion showed little support of the contentions for the appropriation plan. The statistics cited were characterized as misleading, since they do not give expression to the much increased capacity of each unit, whether car or engine. And, in the matter of car-repair costs, even if the cars had not increased in size, the greater tractive capacity of the modern locomotive would cause greater maintenance charge on the cars. It was generally conceded that railway shops were not operated in the best manner, but most of the blame was placed on the practice, dictated by "Wall Street requirements," of reducing working force and shop expense in the summer months, the result of which is, first, to disorganize the working force, and, second, to require the bulk of the repair work to be done with a rush just when all the equipment is most needed. The prompt handling of the autumn traffic is thereby made impossible, and the shop manager is not able to show either effective or economical work. Some of the evils which the author criticized were held to be almost wholly remedied by the piece work plan, when properly administered. Among the speakers were Messrs. W. S. Morris, F. W. Brazier, G. M. Basford, Geo. W. West, W. M. McIntosh, G. R. Henderson and others.

PIKE'S PEAK POLYTECHNIC SOCIETY.—A society of engineers under the preceding name was organized at a meeting held at Colorado Springs, Colo., on Nov. 10, 1904. It is planned to bring together engineers of all classes, civil engineers and surveyors, mining engineers, mechanical engineers, electrical engineers, architects, irrigation and forestry engineers and also geologists, assayer, chemists and others who are studying or practicing along technical lines. Its object is the promotion of intercourse, observation and records in technical subjects, by means of periodical meetings, reading papers, discussions, special investigation into matters of public and technical interest, the publication of such parts of the proceedings as may be deemed expedient, providing the convenience of a library and reading room for its members, and the collection of books and periodicals on techincal subjects. It is also the intention to secure the services of prominent specialists in various technical branches represented by the society, to lecture before the members of the society and invited guests.

The regular meetings are held on the second Saturday of each month in the society's rooms in Coburn Library, on the Colorado College campus, Colorado Springs. The annual dues for resident members are $5 and for non-resident members $3. There are at present 80 members in good standing. The following are the society's officers for 1905: President, Mr. Wm. Strieby; Vice-Presidents, Messrs. B. H. Bryant, L. E. Curtis, and W. F. Douglas; Recording Secretary, Mr. E. A. Sawyer; Corresponding Secretary, Mr. W. D. Waltman; Treasurer, Mr. Ira A. Miller.

Since its organization the society has had papers or lectures as follows:

"Notes on Lightning," by Prof. J. C. Shedd; "The Guanajuato Power Plant," by Mr. L. E. Curtis; "Forestry," by Prof. W. G. M. Stone; "The Panama Canal," by Prof. J. H. Kerr; "Chicago's Transportation Problem," by Mr. Bion J. Arnold; "Underground Waters." by Mr. E. C. Van Diest and Prof. J. H. Kerr; "The Forestry Movement." by Mr. Clyde Leavitt; "French Architecture, 8th to 14th Centuries," by Mr. Nicolaas van den Arend. Any questions in regard to the society or to technical matters in general in Colorado, will be cheerfully answered by the recording secretary or the corresponding secretary. Cor. Secy., W. D. Waltman.